Miscellaneous Paper GL-96-14
May 1996

US Army Corps of Engineers
Waterways Experiment Station

Centrifugal Consolidation Testing of Soils for Classification Purposes

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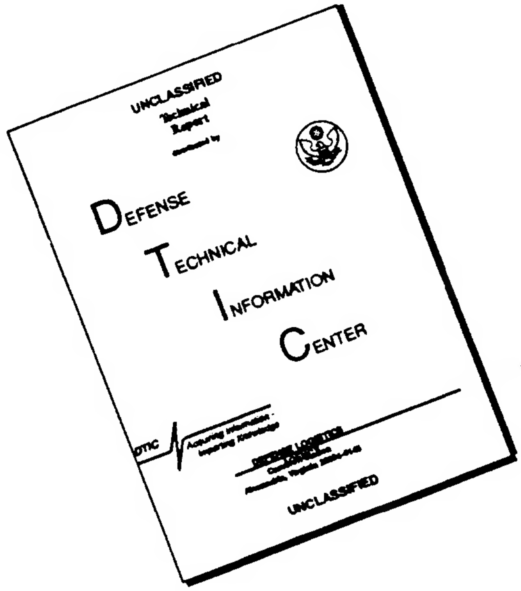

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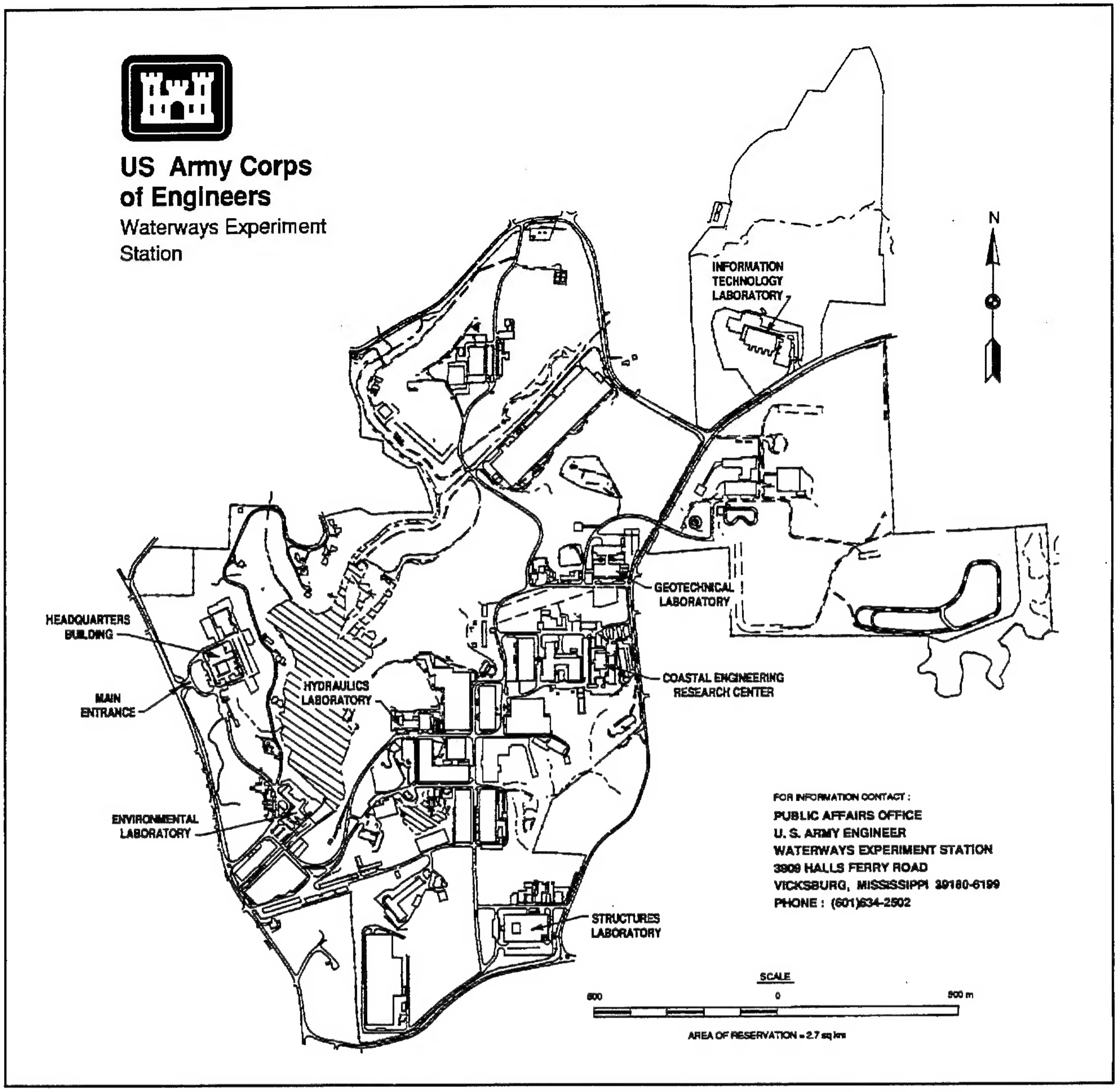

Waterways Experiment Station Cataloging-in-Publication Data

Centrifugal consolidation testing of soils for classification purposes / by John F. Peters ... [et al.] ; prepared for U.S. Army Corps of Engineers.
63 p. : ill. ; 28 cm. -- (Miscellaneous paper ; GL-96-14)
Includes bibliographic references.
1. Soils -- Testing. 2. Soils -- Classification. 3. Centrifuges. 4. Soil consolidation. I. Peters, John F. II. United States. Army. Corps of Engineers. III. U.S. Army Engineer Waterways Experiment Station. IV. Geotechnical Laboratory (U.S. Army Engineer Waterways Experiment Station) V. Series: Miscellaneous paper (U.S. Army Engineer Waterways Experiment Station) ; GL-96-14.
TA7 W34m no.GL-96-14

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Preface

This investigation was funded through the U.S. Army Engineer Waterways Experiment Station (WES) In-House Laboratory Innovative Research (ILIR) program. The research was initially funded for 3 years with the funds from the third year being used for report publication. However, at the end of the second year, funds for the ILIR program were reduced, and this investigation was deleted from the program. Because funds were not available from the ILIR program, publication of the report was delayed.

The purpose of this investigation was to develop a testing procedure for determining soil classification and consolidation properties by use of a centrifuge. A series of centrifuge tests were performed on a group of soils varying from silts to clays to obtain soil properties comparable to those acquired using conventional testing procedures.

The development of the testing program and the testing procedures were performed by Dr. John F. Peters, Ms. Tina L. Holmes, Mr. Daniel A. Leavell, and Ms. Katrina F. Williams (contract student), Soil and Rock Mechanics Division, Geotechnical Laboratory (GL); and Dr. Donald R. Snethen, Oklahoma State University.

The principal investigator of this project was Dr. Peters who worked under the direct supervision of Mr. Gene P. Hale, Chief, Soils Research Center, and under the general supervision of Mr. Clifford L. McAnear, Chief, Soil Mechanics Division, and Dr. William F. Marcuson III, Director, GL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. COL Bruce K. Howard, EN, was Commander.

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Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	25.4	millimeters
pounds (force)	4.448222	newtons
pounds (force) per foot	14.5939	newtons per meter
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6894.757	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
tons (force) per square foot	95.76052	kilopascals

1 Introduction

Soil classification systems such as the Unified Soil Classification System (USCS) (American Society for Testing and Materials (ASTM) 1992) and the American Association of State Highway and Transportation Officials System (AASHTO 1978) are based on physical properties of soils correlated with experience in the use of the soil for a specific purpose, e.g., a highway or airfield subgrade. The basic physical properties used in these use-type classification systems are grain size distribution and plasticity as reflected in the Atterberg limits. The experience bases for the USCS and the AASHTO System were accumulated by Casagrande and Hogentogler, respectively, and were generally limited to subgrade applications. Although the soil characteristics that must be known for proper subgrade design are roughly those needed for other applications, the effectiveness of the classifications for general applications is more limited. In the case of clay soils, which make poor subgrade materials, the classification groups tend to be too broad. For example, a large range of strength and compressibility properties may be attributed to soils that fall under the AASHTO system as CL.

It is well known that the mechanical properties of a clay soil can be summarized from its state of consolidation, a state defined by the prevailing effective stress and void ratio relative to the effective stress-void ratio relationship for virgin compression.[1] The SHANSEP procedure (Ladd and Foote 1974) relates the strength of the clays to overconsolidation ratio (OCR) which is the ratio of the stress in a virgin compression state to the prevailing stress state. The Hvorslev (1937) strength theory and the critical soil mechanics theory (Schofield and Wroth 1968) apply a similar measure of consolidation state.[2] The critical state concept has many uses in soil mechanics including classification of liquefaction potential (used primarily in sands for which the concept of consolidation still applies but

1 In this report, virgin compression refers to soil loaded from an initial as-sedimented state.

2 Hvorslev/critical state theory relates the current stress to the stress in a virgin consolidation state at the prevailing void ratio. OCR compares the prevailing stress to the stress obtained by reloading the soil to the virgin state. Because the recompression index is essentially a constant, both measures provide the same information.

traditionally has not been used), identification of sensitive clays, and as a basis for a multitude of constitutive equations. Leavell and Peters (1987) and Peterson (1990) demonstrate the validity of the concept for partially saturated soils. As a basis for a classification tool, the virgin compression curve provides several important mechanical characteristics.

a. The fundamental compressibility for the normally consolidated state is

$$\Delta e = -C_c \log \frac{p' + \Delta p'}{p'} \tag{1}$$

where

Δe = increment of void ratio

C_c = compression index

p' = effective stress

$\Delta p'$ = increment of effective stress

when e and p' define a point on a plot of effective stress (p') and void ratio (e) that lies on the virgin compression curve and

$$\Delta e = -C_r \log \frac{p' + \Delta p'}{p'} \tag{2}$$

where

C_r = reload coefficient

when e and p' lie to the left of the virgin compression curve. Typically, $C_r \approx 0.2\ C_c$. A stable state (e, p') cannot lie to the right of the virgin compression curve.

b. A basis for computing OCR from which strength can be estimated. Also, knowledge of this position of the virgin curve can improve interpretation of consolidation tests, particularly for soils with high OCR.

c. A basis for identifying soils which are sensitive or those that have properties influenced by cementation. The consolidation state for such soils can lie significantly to the right of the virgin compression curve.

The Atterberg limits can be used to estimate the slope and location of the virgin curve. Schofield (1980) noted that the liquid limit water content corresponds approximately to the water content of a saturated clay in a virgin compression effective stress state of 5 kN/m^2 and the plastic limit

corresponds to stress state of 500 kN/m^2. Based on data assembled by Mayne (1980) and Burland (1990) and data generated by this report, the following general rules apply. The liquid limit corresponds to a virgin stress state of about 5 kN/m^2, and 1.75 to 2 log cycles of stress are required to compress the soil from a water content at liquid limit to that of the plastic limit. The virgin compression state at 100 kPa corresponds to a liquids index of approximately 0.45. The compression index below 100 kPa is approximately 20 percent greater than it is above 100 kPa. The band in which virgin curves fall is relatively small, although soils can still have a wide range in C_c. Thus, as useful as the scheme can be for purposes of classification, Atterberg limits still cannot be used to supplant consolidation tests.

The equipment, training of personnel, and amount of time required to perform the consolidation test make it an expensive test. Also, in current practice, specimens are tested in their natural state in view of the fact that the in situ properties are generally required for most engineering purposes. Consolidation from a reconstituted (slurry) state is difficult in conventional testing equipment, and such testing is too expensive simply to obtain data for classification purposes, regardless of the improvements that could be made in data interpretation. Thus, success in practice of classification schemes using the curve is based on means to determine the curve accurately.

A technique for determining consolidation properties of soils using a laboratory centrifuge was provided by Dr. M. B. Clisby.[1] Using a centrifuge, Dr. Clisby obtained consolidation properties that were in close agreement with results from standard consolidation tests. The time for consolidation by the centrifugal method was 200 min versus 1,440 min per load required for the conventional test.

The investigation presented herein will demonstrate that the soil properties obtained from the consolidation test (i.e., stress, water content) can be obtained through the use of the centrifuge. The water can be removed from the soil by placing the soil in a centrifugal field. The stress applied to the sample during centrifuging can be computed from the centrifugal force, and the volume change can be calculated from the change in the water content. The time required for a centrifugal consolidation is less than that required for the comparable conventional consolidation test, with the differences being greater for high stress levels. For consistent results, the initial water content of the soil should be at its highest stable state for self-weight loading.

1 Personal Communication, 1970, Dr. M. B. Clisby, Mississippi State University, Starkville, MS.

2 Apparatus

The equipment utilized during the development of the test procedure and the subsequent testing program consists of:

a. International Equipment Company (IEC) Centra-7 bench top centrifuge equipped with a four-position horizontal rotor (Figure 1). A stainless steel cup was attached at each of the four positions which supported hard rubber inserts retaining the 1-in.-diam by 4.5-in.-long plastic test tubes (Figure 1). Note: Glass test tubes were used in the early stages of the investigation; however, problems with breakage prior to and during testing resulted in the change to plastic test tubes.

Figure 1. IEC bench top centrifuge with stainless steel cups attached to support rubber inserts for retaining test tubes

b. Mettler Model 1,200 Balance with 0.01 g accuracy.

c. Miscellaneous laboratory items such as ceramic mixing dishes, spatulas, graduated cylinders, and drying ovens (Figure 2).

d. Flexible plastic funnel (Figure 2), typically used for cake decorating, to place the slurried soil mixture in the test tubes.

Materials

A total of 23 soils were used in the investigation. The soils ranged from low plasticity clayey sands (SC) to high plasticity clays (CH). Table 1 shows the basic

physical properties of the 23 soil samples along with the USCS and AASHTO classifications. All physical property tests were conducted using EM 1110-2-1906 (Headquarters, U.S. Army Corps of Engineers 1970) procedures, with the exception of the Bar Linear Shrinkage which was conducted using the Texas Highway Department standard (Tex-107-E) (1970).

Figure 2. Miscellaneous laboratory items

Table1
Summary of Laboratory Test Results

Sample No.	G_s	Atterberg Limits[1]			Bar Linear Shrinkage	Percent Minus No. 200	Classification	
		LL	PL	PI			USCS	AASHTO
1	2.81	39.8	21.0	18.8	13.7	67.2	CL	A-6(11)
2	2.81	30.7	17.7	13.0	8.0	95.0	CL	A-6(12)
3	2.75	45.9	21.8	24.1	17.5	98.4	CL	A-7-6(26)
4	2.76	31.1	19.0	12.1	6.4	95.6	CL	A-6(12)
5	2.79	57.4	22.6	34.8	22.1	91.7	CH	A-7-6(35)
6	2.83	64.4	25.4	39.0	23.1	98.3	CH	A-7-6(35)
7	2.71	31.0	18.5	12.5	8.6	96.0	CL	A-6(12)
8	2.76	29.1	17.2	11.9	9.5	82.5	CL	A-6(8)
9	2.72	26.7	17.4	9.3	8.8	33.2	SC	A-2-4(1)
10	2.71	28.2	16.2	12.0	12.2	56.2	CL	A-6(4)
11	2.83	67.2	29.4	37.8	22.7	90.5	CH	A-7-6(40)
12	2.81	52.2	17.7	34.5	20.9	82.9	CH	A-7-6(30)
13	2.83	64.2	26.1	38.1	20.3	85.2	CH	A-7-6(36)
14	2.75	40.9	19.3	21.6	15.4	92.6	CL	A-7-6(21)
15	2.73	40.8	25.0	15.8	12.2	93.4	CL	A-7-6(17)
16	2.81	61.3	25.3	36.0	22.5	94.8	CH	A-7-6(39)
17	2.75	46.0	24.8	21.2	11.8	67.9	CL	A-7-6(14)
18	2.70	23.7	15.9	7.8	6.6	38.7	SC	A-4(2)
19	2.77	24.3	16.8	7.5	6.4	48.4	SC	A-4(1)
20	2.85	42.1	17.1	25.0	17.7	65.1	CL	A-7-6(14)
21	2.70	31.8	16.2	15.6	13.7	83.7	CL	A-6(12)
22	2.78	48.4	18.8	29.6	17.9	87.8	CL	A-7-6(28)
23	2.80	50.9	19.8	31.1	19.5	91.9	CH	A-7-6(30)

[1] LL = liquid limit, PL = plastic limit, PI = plasticity index.

Calculations

For saturated soils, the void ratio is directly related to the water content. Specifically,

$$Se = wG_s \tag{3}$$

where

S = degree of saturation

e = void ratio

w = water content

G_s = specific gravity of soil particles

The stress on the soil sample is a function of the rotational speed of the centrifuge and can be quantified using angular acceleration on the soil mass at the lower third point in the soil sample. The distances and dimensions for the soil sample and test tube configuration as it rotates in the centrifuge are shown in Figure 3. The effective stress at the lower third point of the soil sample at 100 percent consolidation is given by the following relationship:

$$\sigma'_v = 1.42 \times 10^5 \, (rpm)^2 \, (\gamma_b)\left(r^2 - r_o^2\right) \tag{4}$$

where

σ'_v = effective consolidation (vertical) stress

rpm = rotational speed of the centrifuge

γ_b = buoyant unit weight of the soil, pcf

r = distance from the center of the rotation to the lower third point of the soil sample, inches

r_o = distance from the center of rotation to the top of the soil sample, inches

For the configuration shown in Figure 3, r = 7.2 in., and r_o = 6.7 in. The effective consolidation stress in pounds per square inch is:

$$\sigma'_v = 9.869 \times 10^5 \left(\frac{\gamma_b}{1{,}728}\right)(rpm)^2 \tag{5}$$

Using the void ratio calculated from the measured water contents obtained in the selected force field (i.e., *rpm* and time) and the effective consolidation stress at the respective *rpm*, a void ratio versus effective consolidation stress curve can be plotted.

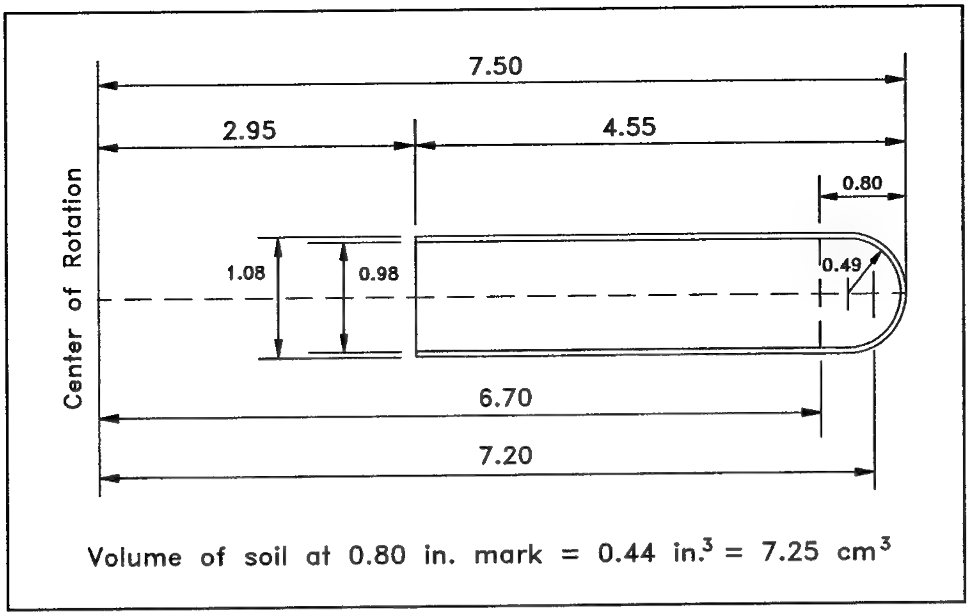

Figure 3. Configurations and dimensions of test tube and soil sample, inches

Procedures

The centrifugal consolidation test procedure involves exposing a soil sample to varying centrifugal forces for different lengths of time and determining the deformation (consolidation) of the sample by weighing the extracted water and measuring the height at the various time intervals. Testing procedures were varied during the early portion of the investigation to determine which procedure provided the best results. Three soils, one low plasticity (soil sample 18) and two high plasticity (soil samples 12 and 20), were used during the procedure variations.

In the first test, soil sample 18 was mixed to a water content slightly above the liquid limit and placed in test tubes to a depth of 1 in. The specimens were tested at various speeds between 500 and 2,500 *rpm* for 15 min at each speed without the sample's being inundated. The tests resulted in a series of void ratio versus effective consolidation stress curves (Figure 4) with an average compression index of 0.346. One significant problem noted during each test was the drying of the upper surface of the specimen, particularly at the higher speeds.

During the second test, soil sample 18 was mixed to a water content slightly above the liquid limit and placed in test tubes to a depth of 1.5 in. Two of the four samples were covered with 10 cc of water to maintain saturation. The tops of all four test tubes were covered with plastic film to minimize evaporation. Rather than varying the rotational speed during the test, specific rotational speeds were selected which provided specific centrifugal forces based on the buoyant unit weights of the specimens. The results were not consistent because primary consolidation was never achieved during centrifuging at the various speeds, even with as much as

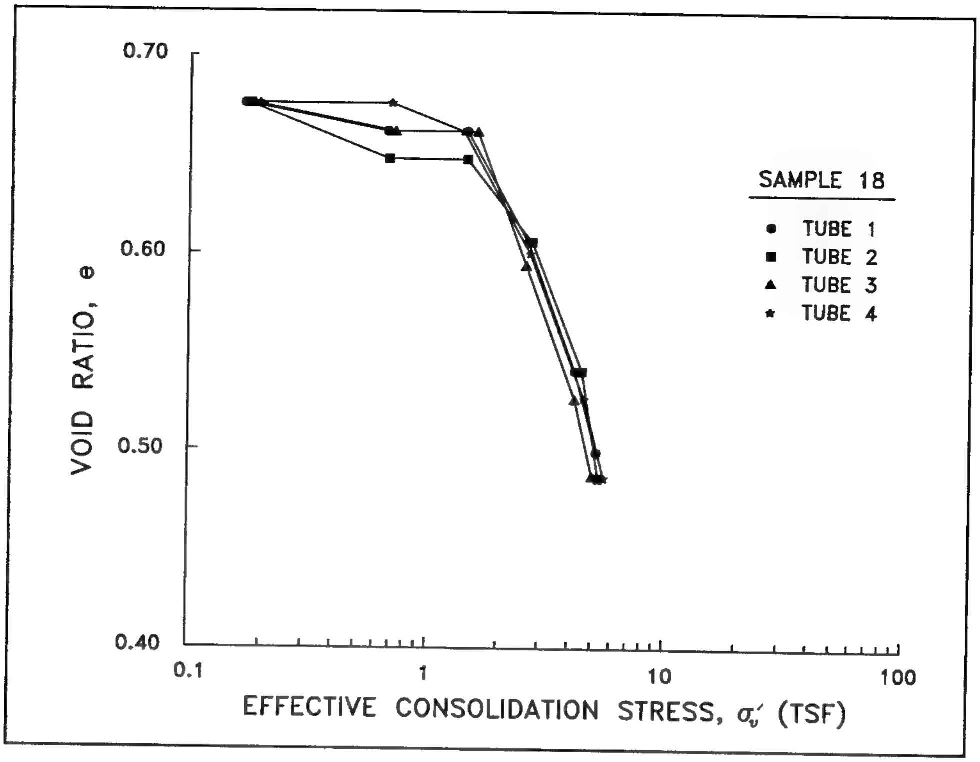

Figure 4. Void ratio versus effective consolidation stress for soil sample 18

6 hr cumulated spin time. The primary reason for this inconsistency was that the sample was too thick.

In the third test, both soil samples 18 and 20 were mixed to water contents slightly above the liquid limit and placed in test tubes (two specimens of each soil) to a depth 0.75 in. The specimens were covered with 10 cc of water and tested at five speeds for various cumulated spin times. The void ratios were determined from the changes in water content based on the changes in weight of the specimens at various times. Effective stresses were calculated using the buoyant unit weight and specific *rpm*. The results of the tests are shown in Figures 5 and 6 (soil sample 18) and in Figures 7 and 8 (soil sample 20). The irregular curve for the 2- and 6-hr cumulative spin time for tube No. 1 could not be explained. For the low plasticity soil (soil sample 18), there appears to be minimal effect of spin time on the compression index, particularly for tube No. 2. For the high plasticity soil (soil 20), the shapes of the various curves are similar, with some small variation in the slopes of the curves. The small variation in the compression index does not significantly affect the results of this investigation since the primary purpose was to establish a procedure from which classification data could be derived.

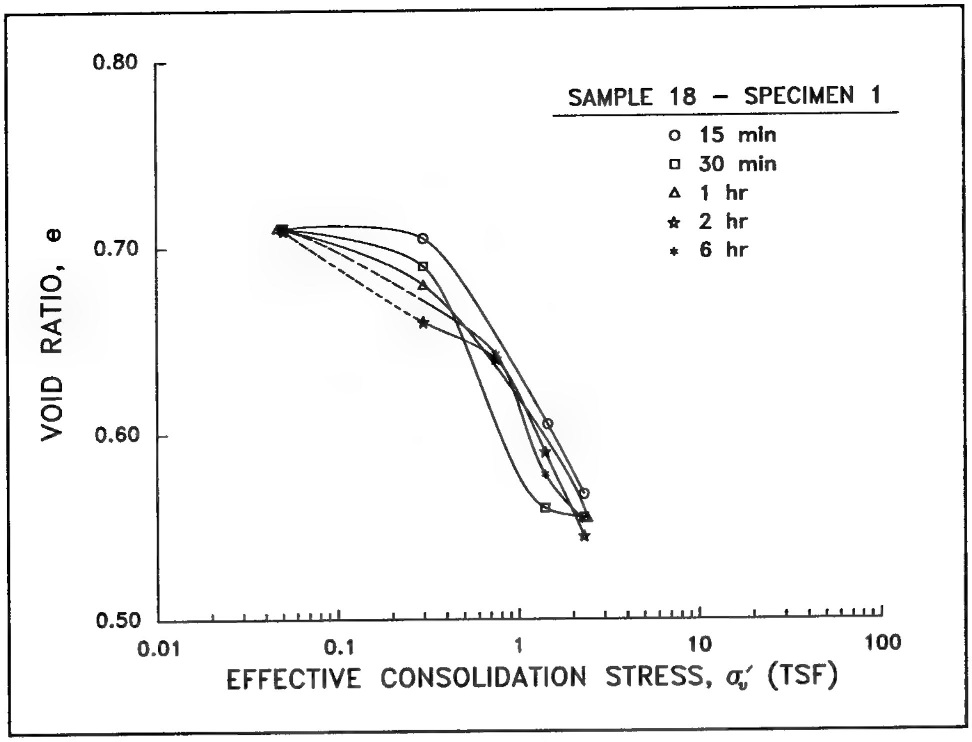

Figure 5. Void ratio versus effective consolidation stress curves for various cumulative spin times, sample 18, specimen 1

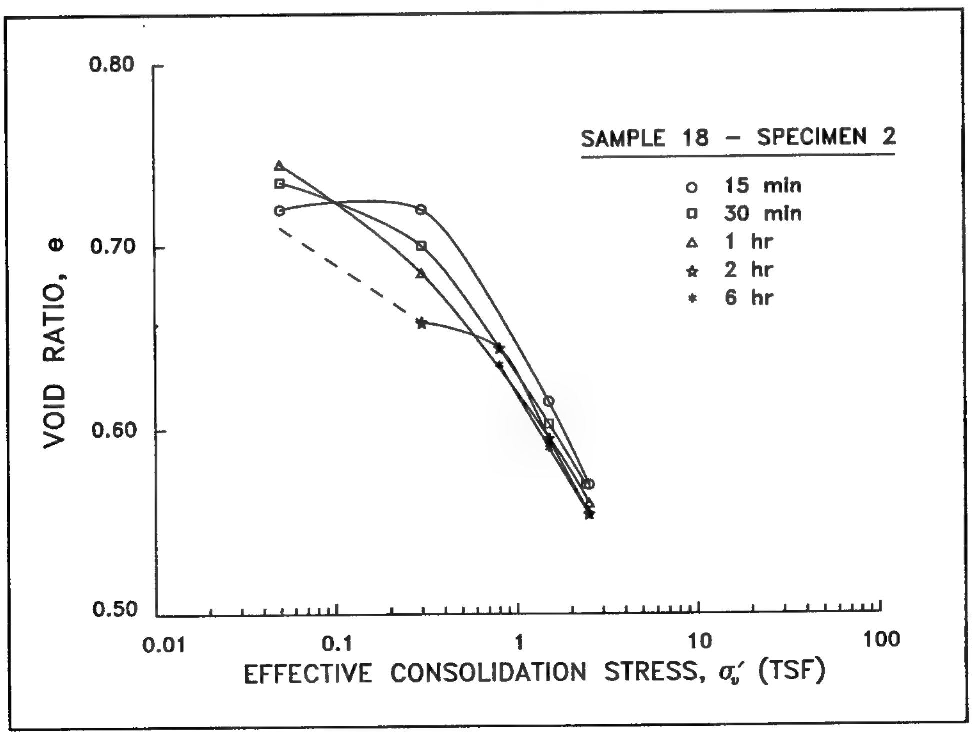

Figure 6. Void ratio versus effective consolidation stress curves for various cumulative spin times, sample 18, specimen 2

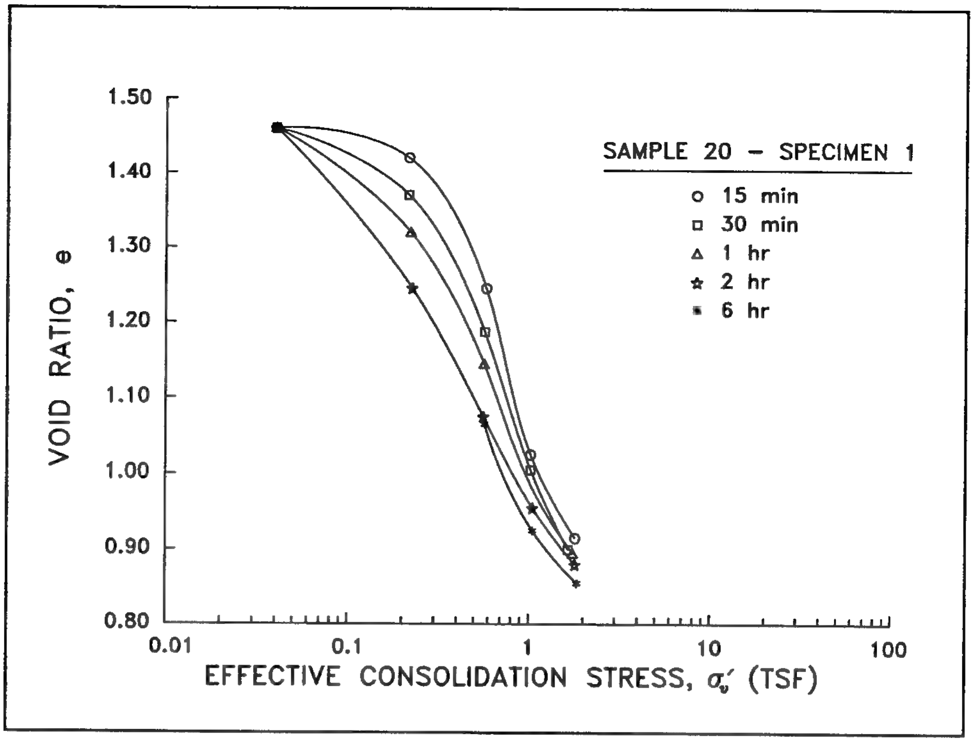

Figure 7. Void ratio versus effective consolidation stress curves for various cumulative spin times, sample 20, specimen 1

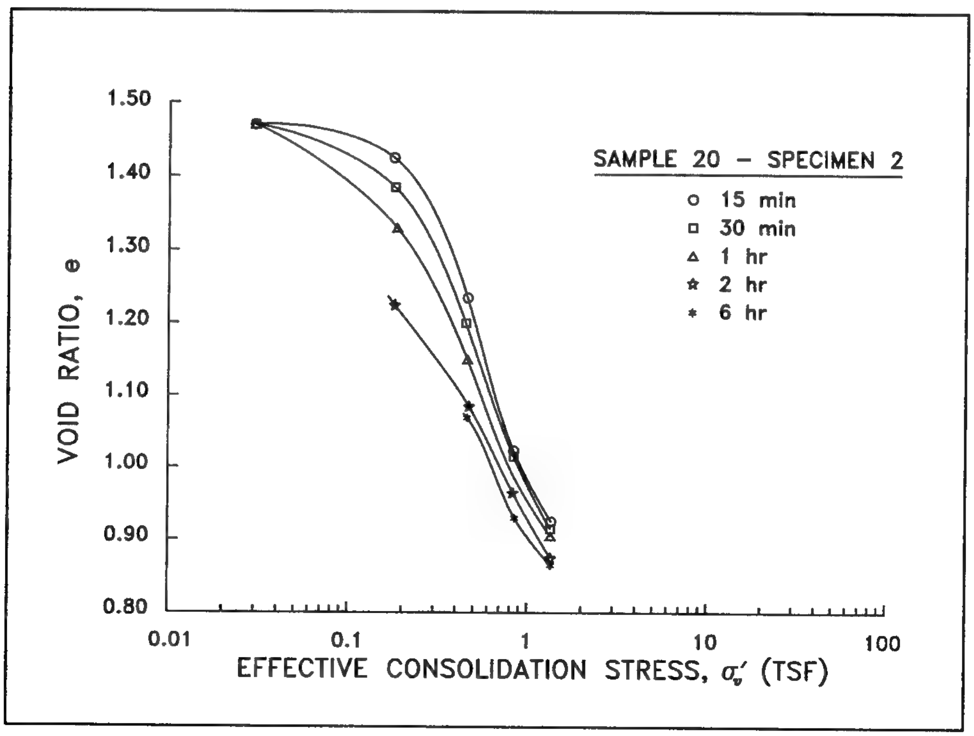

Figure 8. Void ratio versus effective consolidation stress curves for various cumulative spin times, sample 20, specimen 2

The results of these preliminary tests basically determined the specimen thickness and spin times that would be used in the subsequent testing program; that is, a specimen thickness or depth of 0.75 in. and a spin time at each *rpm* of 1 hr. To further verify the spin time selection during actual testing, two cumulated spin times, 0.5 and 1.0 hr, were used on 20 of the samples. As will be discussed later, the difference in the results obtained using the two spin times is not significant.

Additional questions arose concerning the centrifuge testing procedure; specifically, what effect does a delay in the test have on the results (e.g., an overnight delay as might be expected during routine use of the test procedure) and what effect does using the same specimen throughout the test have on the results? The first question is basically directed at rebound and saturation or, more specifically, partial saturation (i.e., air coming out of solution after the sample has been subjected to a centrifugal force field). It was theorized that the presence of air in the soil matrix would affect the time to achieve primary consolidation. The second question essentially extended the first question (i.e., delay in test necessitated by weighing the samples for water content change) as well as addressing the effect on the compression curve.

To address these questions, some additional procedural variation tests were conducted, specifically:

a. Centrifugal consolidation tests were run using the same specimen which was run at the first *rpm* (i.e., 500) then a 24-hr delay was used before completion of the test. During the delay, duplicate specimens were stored with and without water on the soil. After the delay the specimens were tested through the selected *rpms* (i.e., 1,000, 1,500, 2,000, 2,500, 1,500, 500). Parallel tests were run using no delay in the test sequence.

b. Centrifugal consolidation tests were run using seven specimens for each soil sample using 1-hr spin time at each *rpm* and cumulating the time for each specimen.

c. Centrifugal consolidation tests were run using seven specimens for each soil sample using only an hour spin time at each *rpm*.

All three test series were run on a high plasticity soil (sample 12) and a low plasticity soil (sample 18). The results of the additional procedural variation are presented in Figures 9-13 and 14-18 for soil samples 12 and 18, respectively. Figures 9-11 show a minimal effect of the delay in testing sequence and essentially no effect of storing the samples during the delay with or without water. The specimens tested without a delay in the sequence (Figure 11) exhibited a slightly lower average value of C_c as compared to the specimens with the delay. It is apparent from Figures 12 and 13 that the use of multiple specimens, whether cumulating spin time or not, provides inconsistent results. Similar behavior was demonstrated by soil sample 18 with one minor exception shown in Figure 15 which is

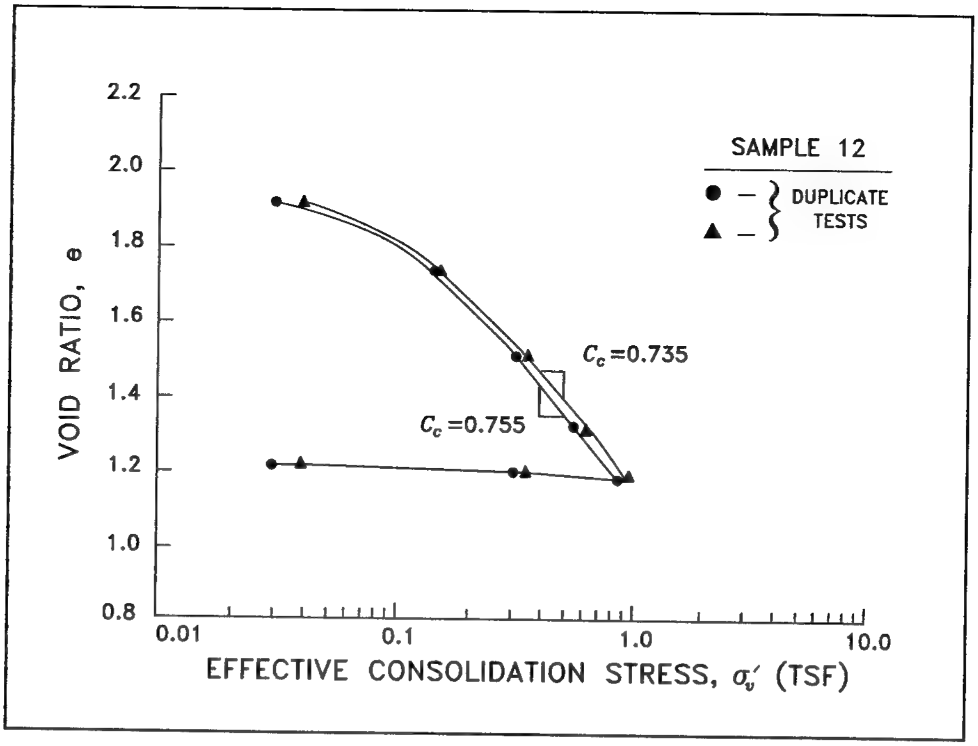

Figure 9. Sample 12, 1-hr spin time (duplicate samples with 24-hr delay between 500 and 1,000 *rpm*. No water on sample during delay)

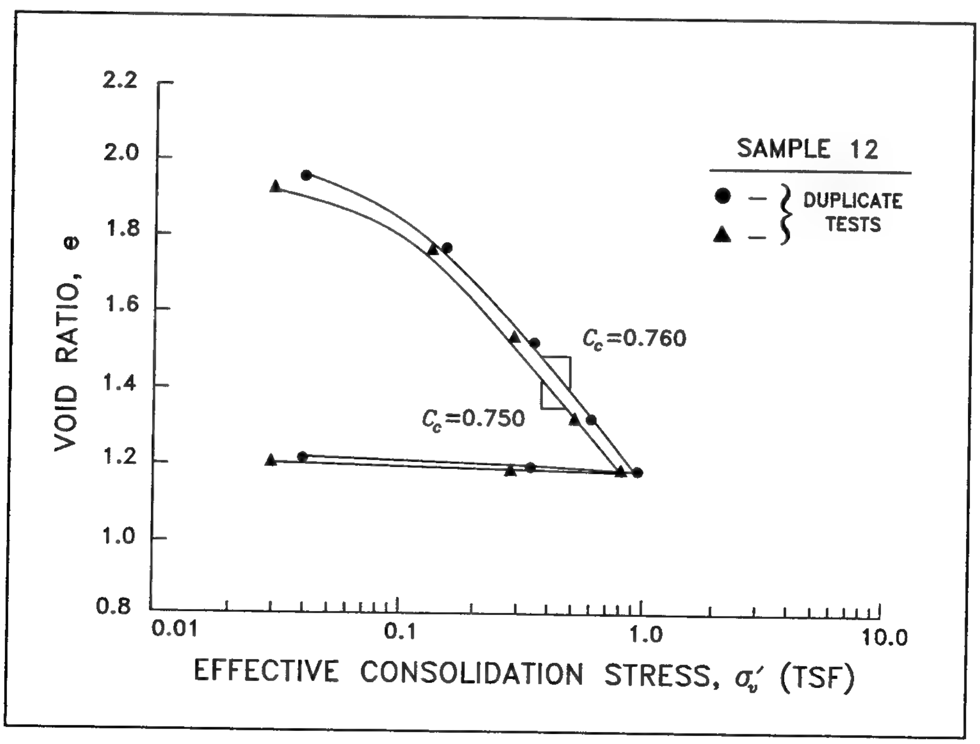

Figure 10. Sample 12, 1-hr spin time (duplicate between 500 and 1,000 *rpm*. Water on sample during delay)

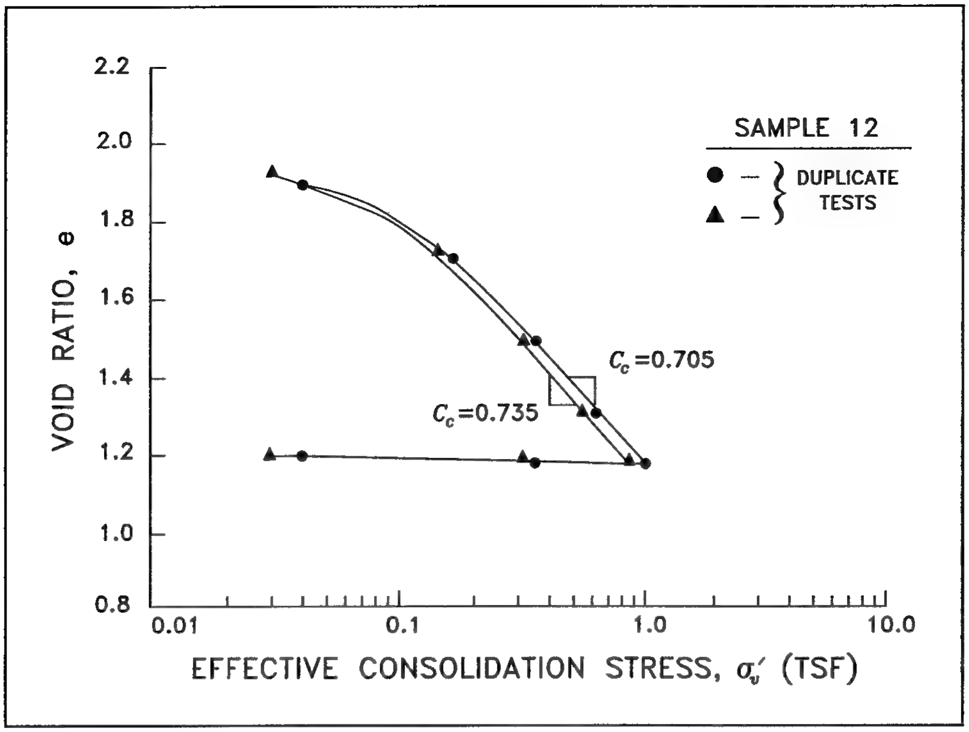

Figure 11. Sample 12, 1-hr spin time (duplicate samples without delay)

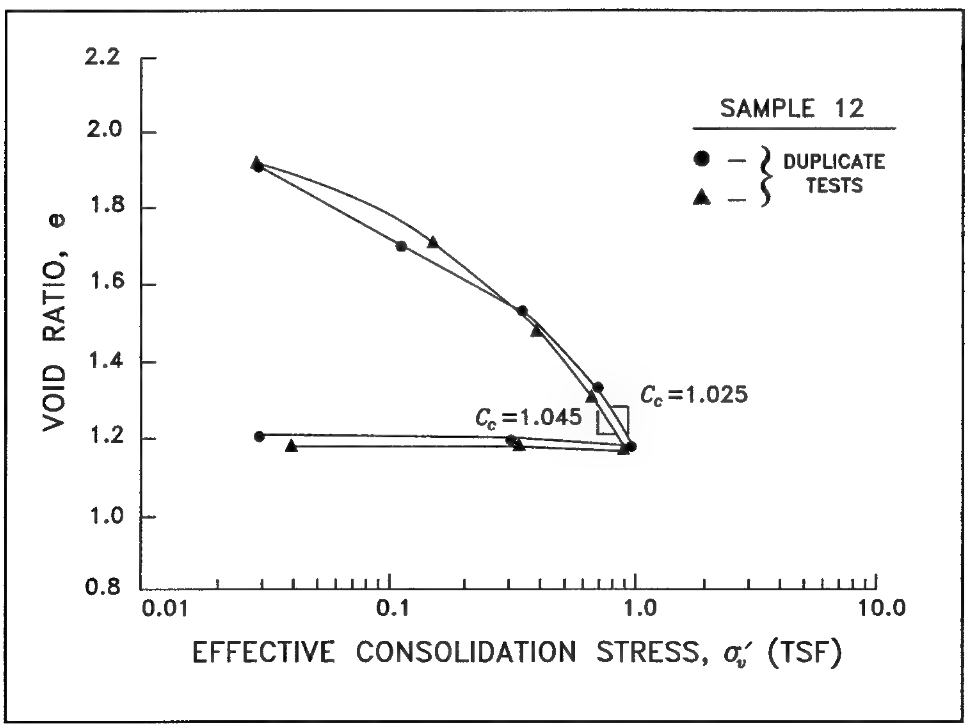

Figure 12. Sample 12, 1-hr spin time (duplicate samples using seven specimens for each and cumulative spin time)

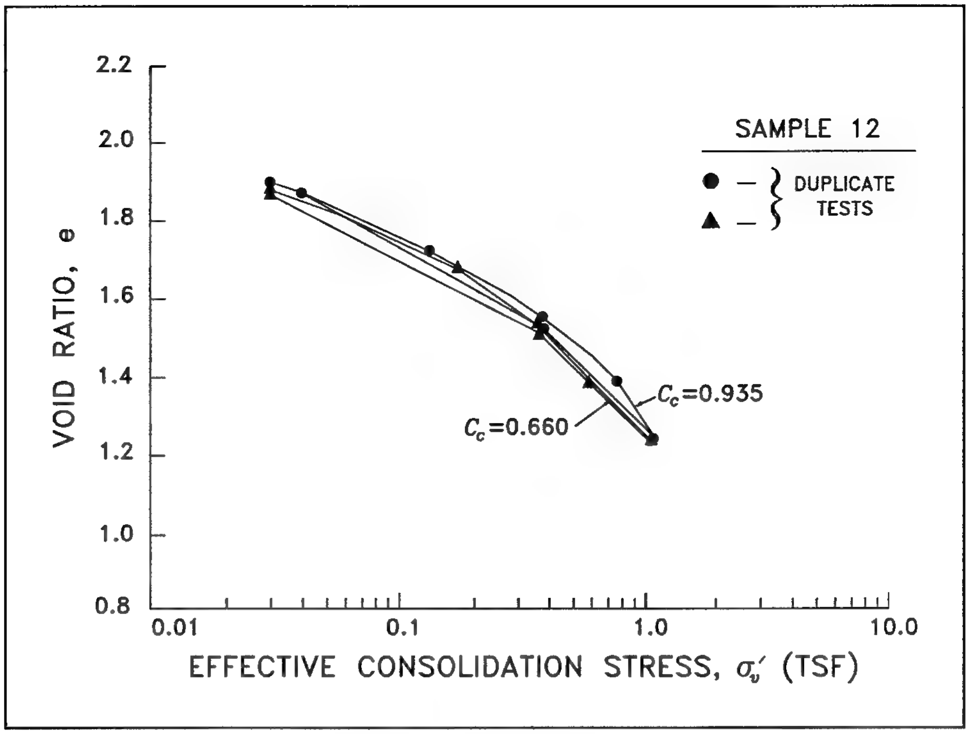

Figure 13. Sample 12, 1-hr spin time (duplicate samples using seven specimens for each and 1-hr spin time for each *rpm*)

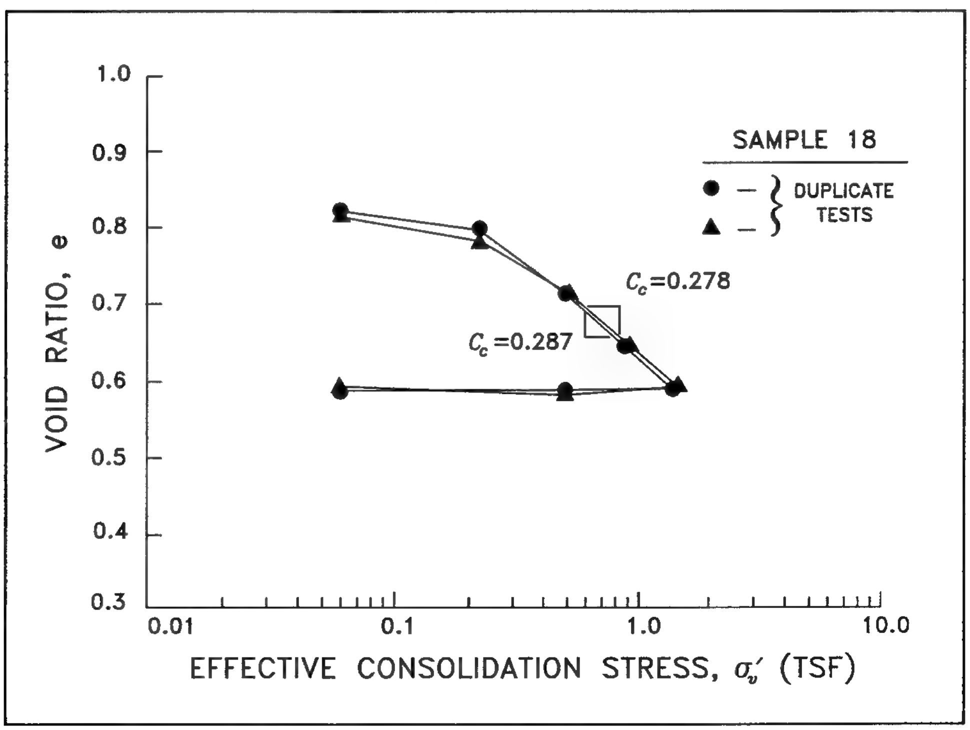

Figure 14. Sample 18, 1-hr spin time (duplicate samples with 24-hr delay between 500 and 1,000 *rpm*. No water on sample during delay)

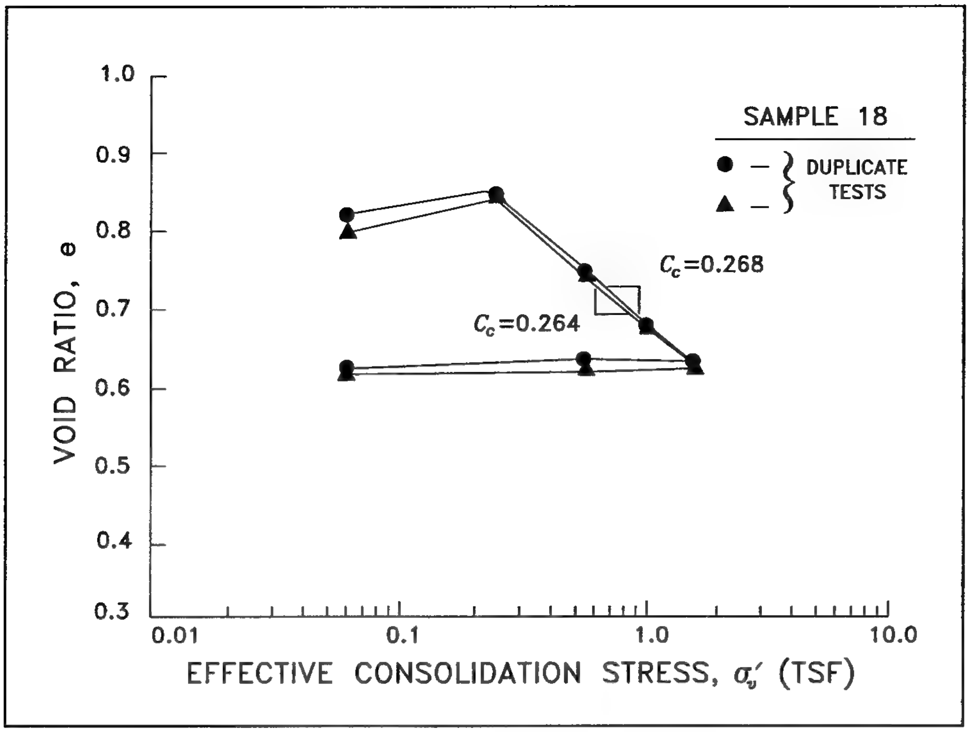

Figure 15. Sample 18, 1-hr spin time (duplicate samples with 24-hr delay between 500 and 1,000 *rpm*. Water on sample during delay)

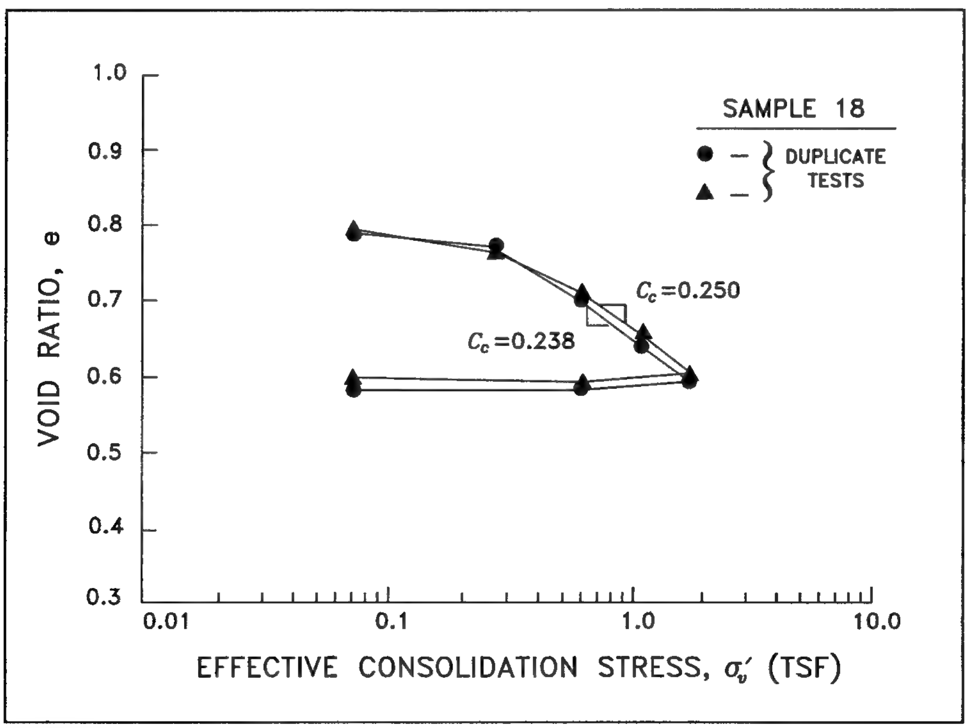

Figure 16. Sample 18, 1-hr spin time (duplicate samples without delay)

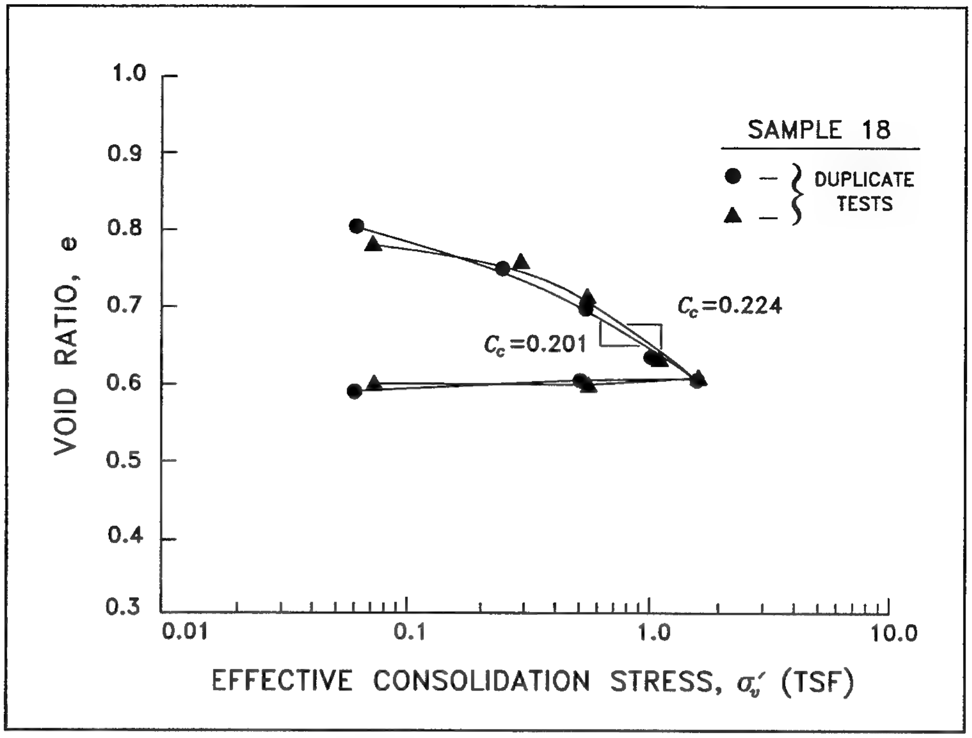

Figure 17. Sample 18, 1-hr spin time (duplicate samples using seven specimens for each and cumulative spin times)

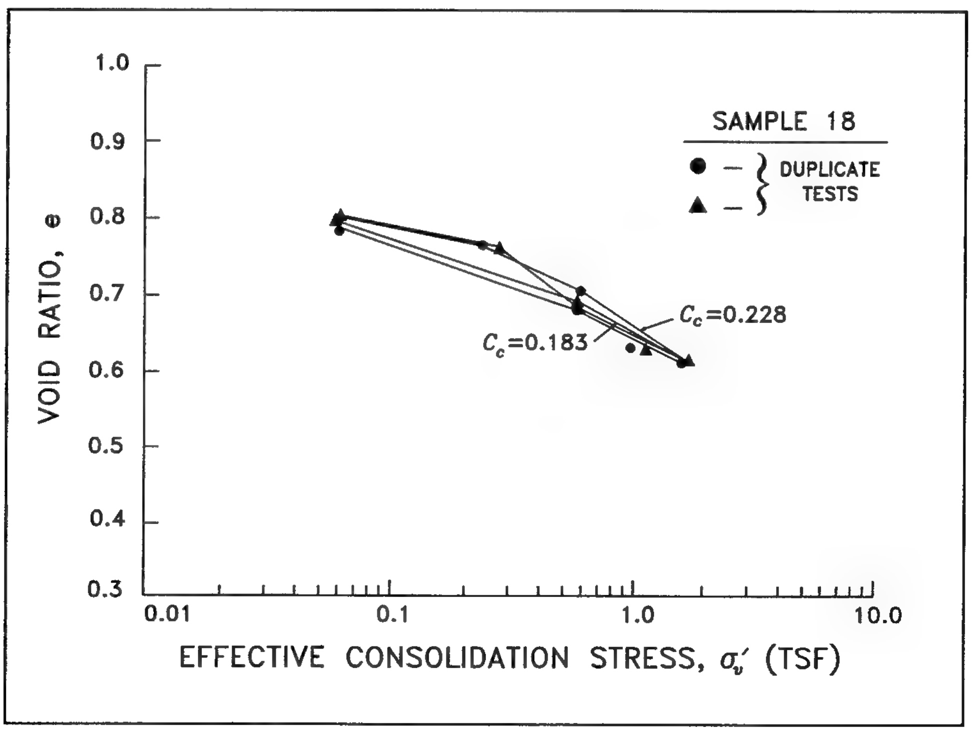

Figure 18. Sample 18, 1-hr spin time (duplicate samples using seven specimens for each and 1-hr spin times for each *rpm*)

an increase in void ratio following the 24-hr delay. This was the basis for the question on partial saturation due to air coming out of solution.

After careful evaluation of the results of the procedure variation tests, the following procedure was selected and used for the testing program:

a. The soil samples were mixed with distilled water to a water content at or slightly above the liquid limit (Figure 19) and allowed to hydrate at least 24 hr. The consistency criteria used for soils was the water content at which a groove cut by drawing the flat side of a small spatula through the soil would flow closed under two or three gentle taps on the palm of the hand.

b. The soil mixture was placed in plastic test tubes using the flexible plastic funnel (Figures 20 and 21). The depth of the soil mixture was extended above the calibration mark (Figure 22). The excess soil was removed using a square-tipped spatula, and the sides of the test tube were wiped clean (Figure 23). The test tube and soil mixture were weighed to establish initial water content and density conditions.

c. Ten cubic centimeters of distilled water were carefully placed over the soil specimen (Figure 24), and the test tube was sealed with plastic film (Figure 25).

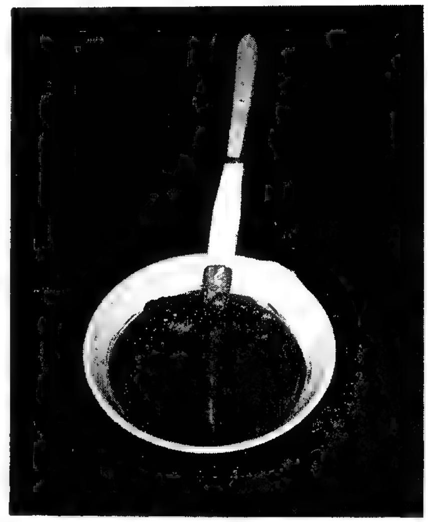

Figure 19. Soil with a water content of slightly above the liquid limit

d. The samples were placed in the centrifuge, and the speed was adjusted to 500 *rpm*. After 30 min spin time, the test tubes were removed, and the water was poured into a temporary container. The sides of the test tube were wiped dry and the test tube and soil weighed to the nearest 0.01 g (Figure 26). The water was poured back in the test tube, the plastic film replaced, and the specimens were spun for an additional 30 min. The water was again removed, the test tube dried, and the test tube and soil weighed to the nearest 0.01 g.

e. After returning the water to the test tube and replacing the plastic film, step *d* was repeated for four additional centrifuge speeds (1,000, 1,500, 2,000, and 2,500 *rpm*).

f. After collecting the centrifuge consolidation data and discarding the water, the test tubes and soil were weighed and dried to a constant weight so the initial and final water contents could be determined.

g. Using procedures outlined earlier in this report, the void ratio and effective stress conditions were calculated and the *e* versus log *p* curves were plotted.

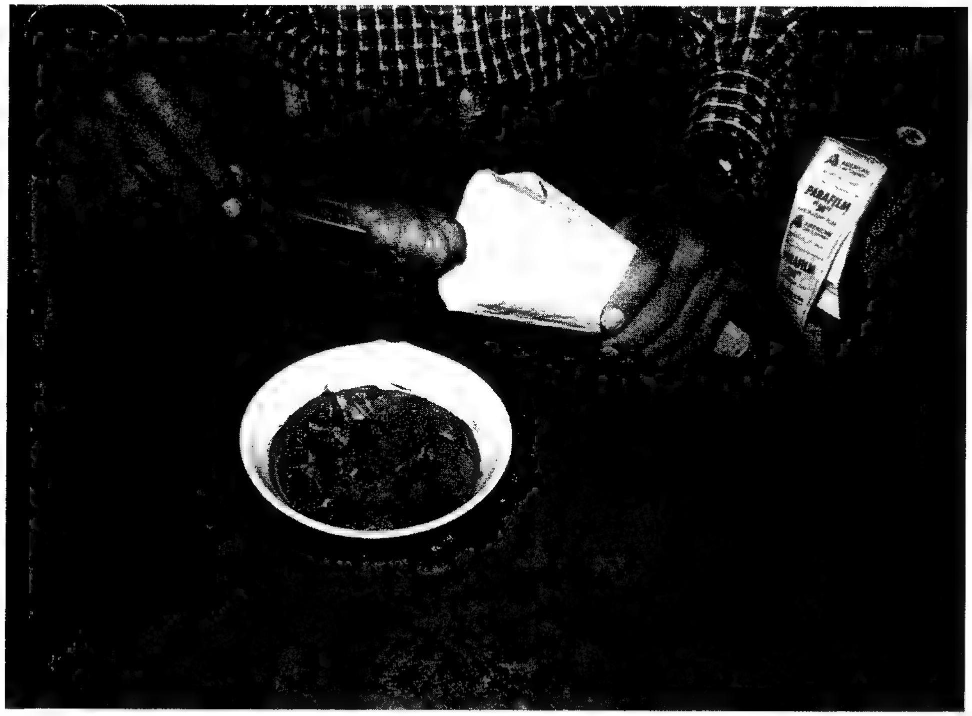

Figure 20. Soil being placed in a flexible plastic funnel

Figure 21. Soil being placed in test tube through the funnel

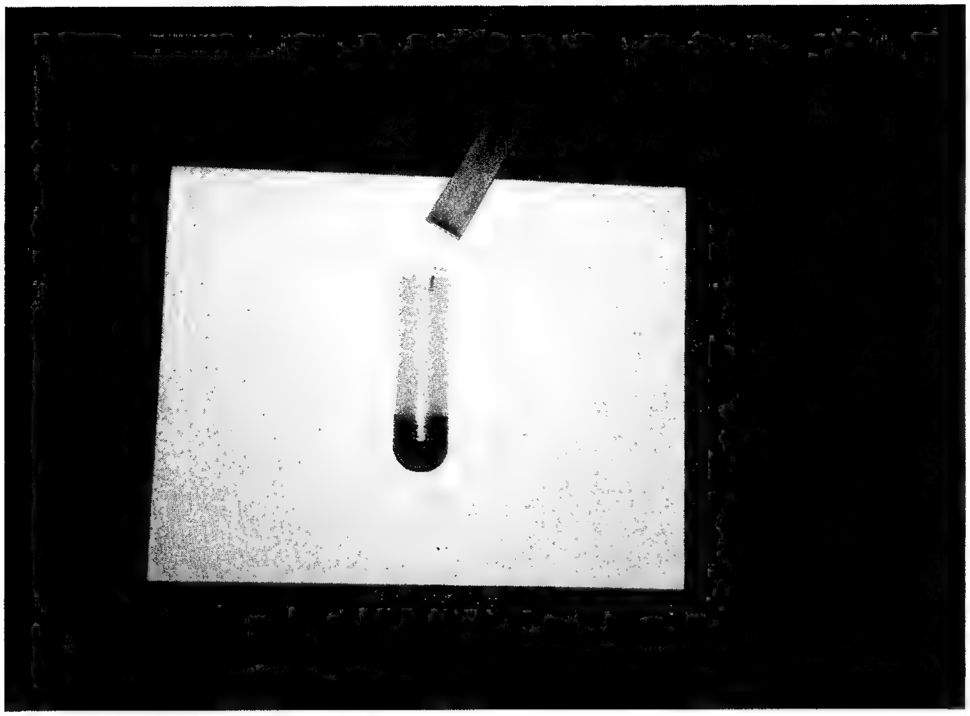

Figure 22. Soil depth extended above the calibration mark

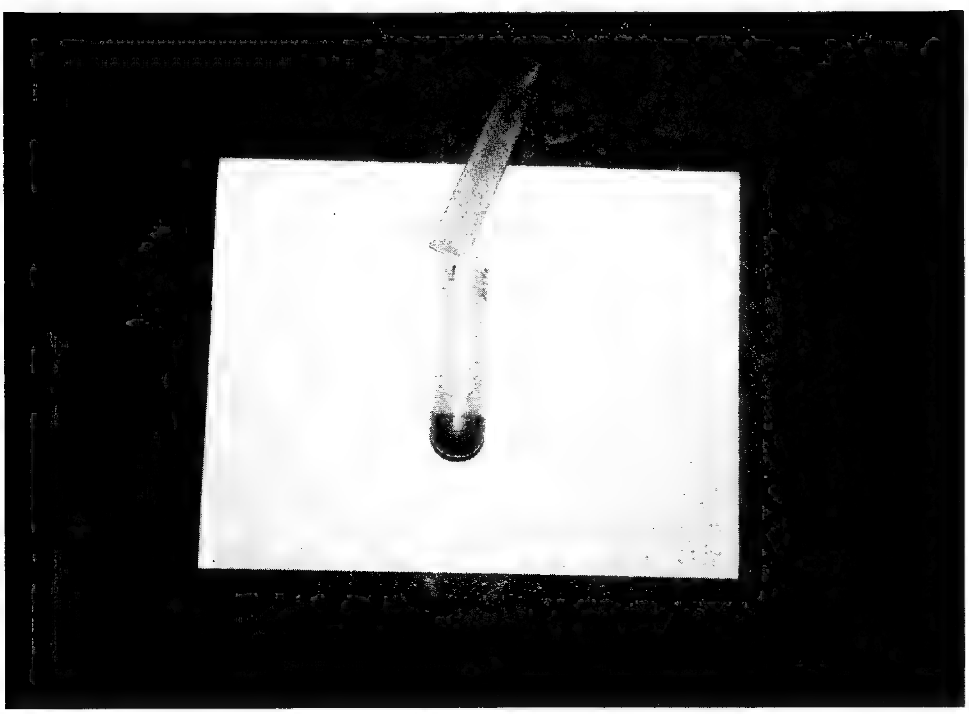

Figure 23. Excessive soil removed and test tube cleaned

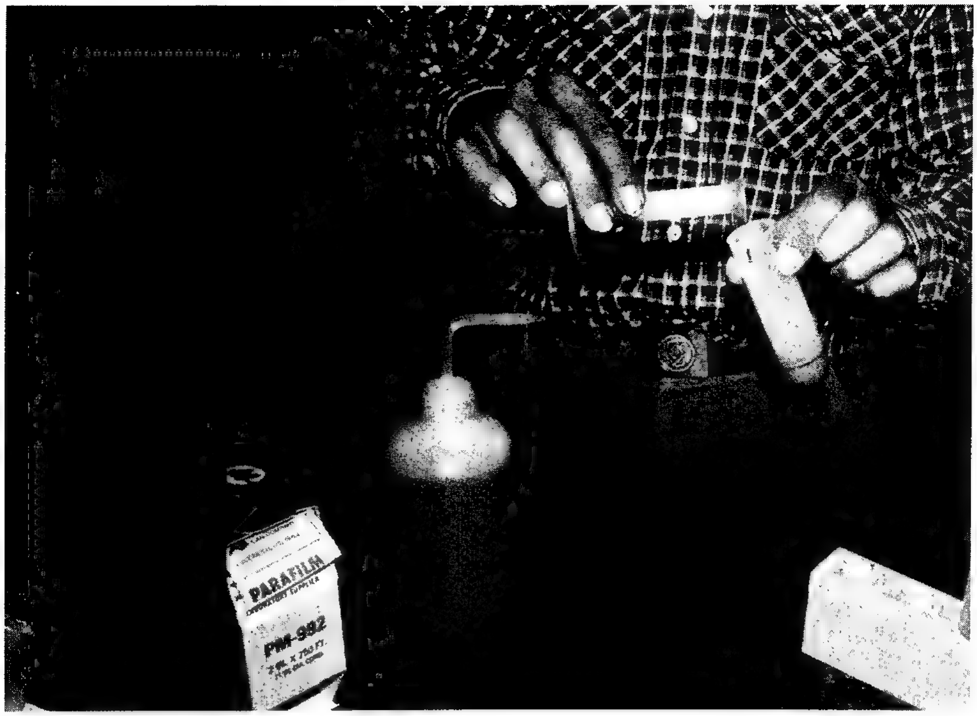

Figure 24. Distilled water placed over the soil specimen

Figure 25. Test tube sealed with plastic film

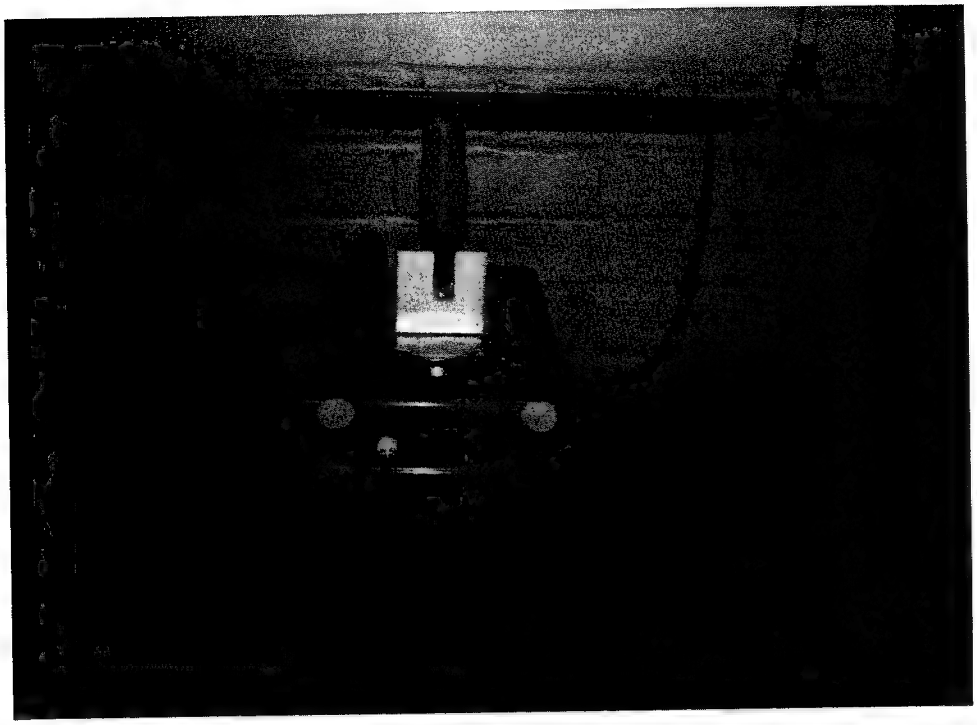

Figure 26. Test tube and soil weighed to the nearest 0.01 g

3 Results

The results of the centrifuge consolidation tests on 23 fine-grained soils were reduced to void ratio and effective stress data and plotted as *e* versus log *p* curves for each soil sample. Duplicates of all specimens were run. The resulting *e* versus log *p* curves are presented in Table 2 along with initial and final water contents and void ratios for each soil sample.

Table 2
Summary of Centrifuge Test Results

Sample No.	Initial Conditions		Compression Index and Void Ratio: 0.5-hr Spin Time		Compression Index and Void Ratio: 1.0-hr Spin Time		Final Conditions	
	w, percent	*e*	C_c	*e* @ 1, tsf	C_c	*e* @ 1, tsf	*w*, percent	*e*
1	46.45	1.305	0.369	0.787	0.294	0.765	27.82	0.782
2	36.91	1.000	0.295	0.709	0.266	0.683	25.17	0.697
3	51.89	1.427	0.389	0.925	0.330	0.900	32.22	0.886
4	39.69	1.095	0.217	0.661	0.182	0.640	23.55	0.650
5	57.99	1.618	0.757	1.205	0.706	1.150	46.27	1.291
6	71.92	2.035	0.667	1.240	0.607	1.198	46.66	1.320
7	35.85	0.972	0.206	0.648	0.199	0.627	23.77	0.644
8	33.38	0.921	0.169	0.638	0.162	0.621	22.99	0.635
9	31.74	0.863	0.225	0.527	0.183	0.588	20.35	0.554
10	33.98	0.921	0.220	0.611	0.161	0.608	21.51	0.583
11	65.35	1.849	0.432	1.382	0.425	1.382	49.41	1.398
12	60.84	1.710	0.516	1.036	0.425	1.039	38.73	1.088
13	65.94	1.866	0.501	1.285	0.493	1.258	47.37	1.341
14	42.87	1.179	0.344	0.827	0.336	0.804	29.34	0.807
15	43.66	1.192	0.301	0.853	0.301	0.834	31.50	0.860
16	60.77	1.708	0.503	1.121	0.468	1.099	41.61	1.169
17	54.47	1.498	0.328	1.060	0.302	1.041	33.04	0.909
18	26.23	0.708	0.266	0.568	0.258	0.551	20.27	0.547
19	26.86	0.744	0.117	0.531	0.100	0.524	19.08	0.529
20	50.19	1.430	0.628	0.911	0.599	0.863	32.19	0.917
21	41.33	1.116	—	—	0.225	0.694	24.85	0.674
22	50.80	1.412	—	—	0.391	0.953	33.97	0.932
23	55.37	1.551	—	—	0.569	1.138	40.47	1.128

Comparisons between Tables 1 and 2 show that the consistency criteria for initial water content conditions worked reasonably well since the initial water contents varied between 2 and 8 percent above the liquid limit for 21 of the 23 soils. Closer control could obviously be obtained if the liquid limit or some specified increment above the liquid limit were required; however, this would basically defeat the purpose of enhancing the test as an alternative classification system. The load increment will be discussed later in the discussion of results. An alternative procedure would be to initiate the test from an as-sedimented state, a procedure that would not only provide a consistent initial condition, but also lead to a more representative consolidation curve.

Compression indexes were generally lower for the 1-hr spin time as compared to the 30-min spin time. The differences were smaller for the lower plasticity soils and larger for the higher plasticity soils. The determination of the "correct" compression index is extremely difficult since this would require some standard for comparison, for example, the compression index for undisturbed samples, preferably run in a large-scale centrifuge consolidation test. Comparisons of the compression index between the centrifuge consolidation test on a slurry-mixed sample and the conventional consolidation test on an undisturbed sample were not very good, as noted in Figures 27-29. C_c values from conventional consolidation tests were from 33 to 67 percent lower than those obtained from the centrifuge consolidation tests. As the plasticity of the soil increased, so did the difference between the C_c values from the two test methods.

The secondary purpose of the investigation was to evaluate the possibility of enhancing current soil classification systems with "consolidation data" obtained using the centrifuge procedure. Since limited information was developed on correlations between conventional and centrifugal consolidation tests (Figures 27-29), the basis for enhancement falls primarily to how well the developed data correlate or support the existing systems, specifically the USCS. Using the liquid limit and plasticity index to plot against the compression index as shown in Figures 30 and 31, respectively, a reasonable correlation is evident. The correlation does not appear to be linear, although one could certainly "force" a linear band over the majority of the data. Even though an apparent correlation does exist between Atterberg limits and the compression index, it does not appear to be significant enough to warrant a statistical investigation without more supporting consolidation data (either centrifugal or conventional).

An interesting variation of the compression index data was noted when the C_c values were plotted on the USCS plasticity chart (Figure 32). With the exception of sample 20, the data plot in three somewhat distinct groupings. For example, C_c values less than 0.3 grouped in the lower plasticity area of the CL category; C_c values between 0.3 and 0.5 grouped in the higher plasticity area of the CL category; and C_c values greater than 0.45 grouped in the CH category. Looking at this grouping optimistically, it does support the general correlations previously described and establishes a limited subcategorization from which compressibility characteristics could

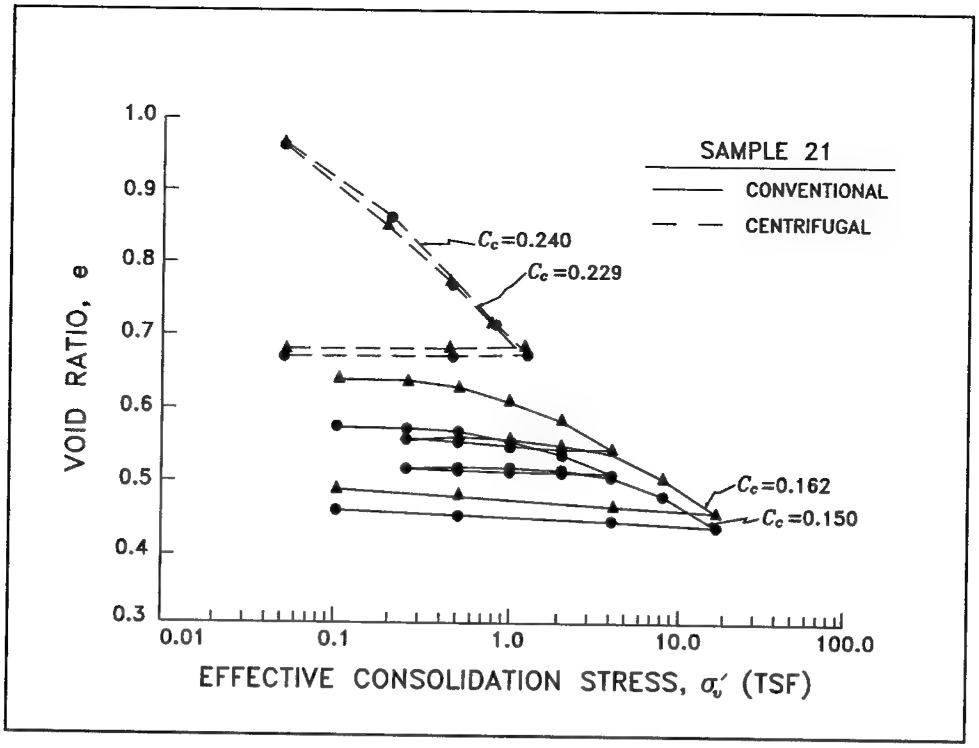

Figure 27. Soil sample 21, comparison of conventional (solid line) and centrifugal (dashed line) consolidation tests

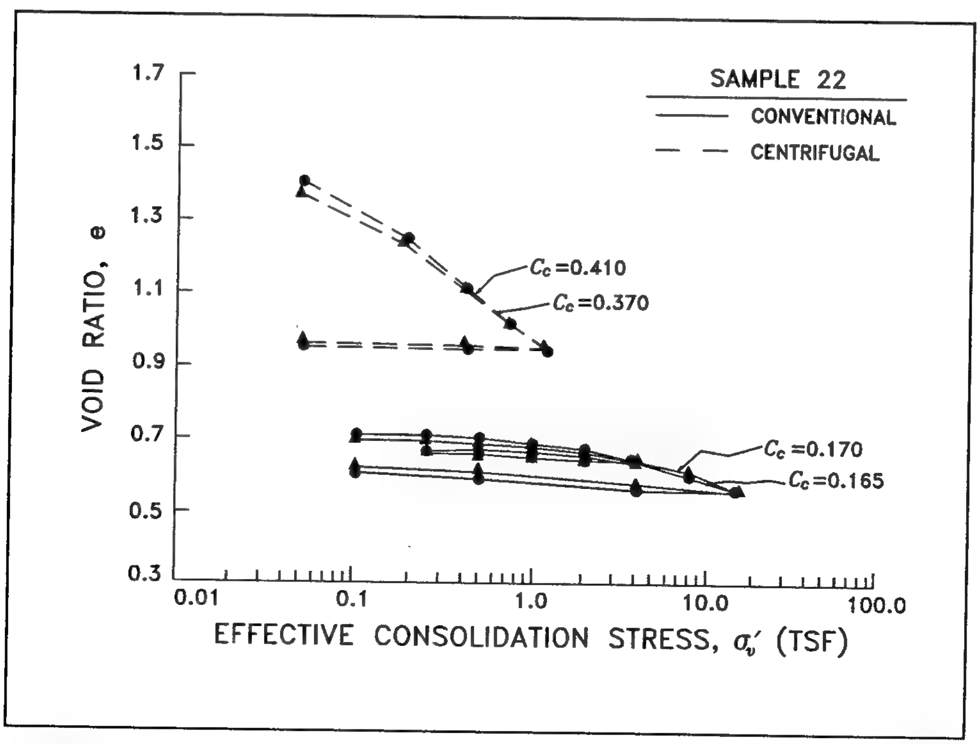

Figure 28. Soil sample 22, comparison of conventional (solid line) and centrifugal (dashed line) consolidation tests

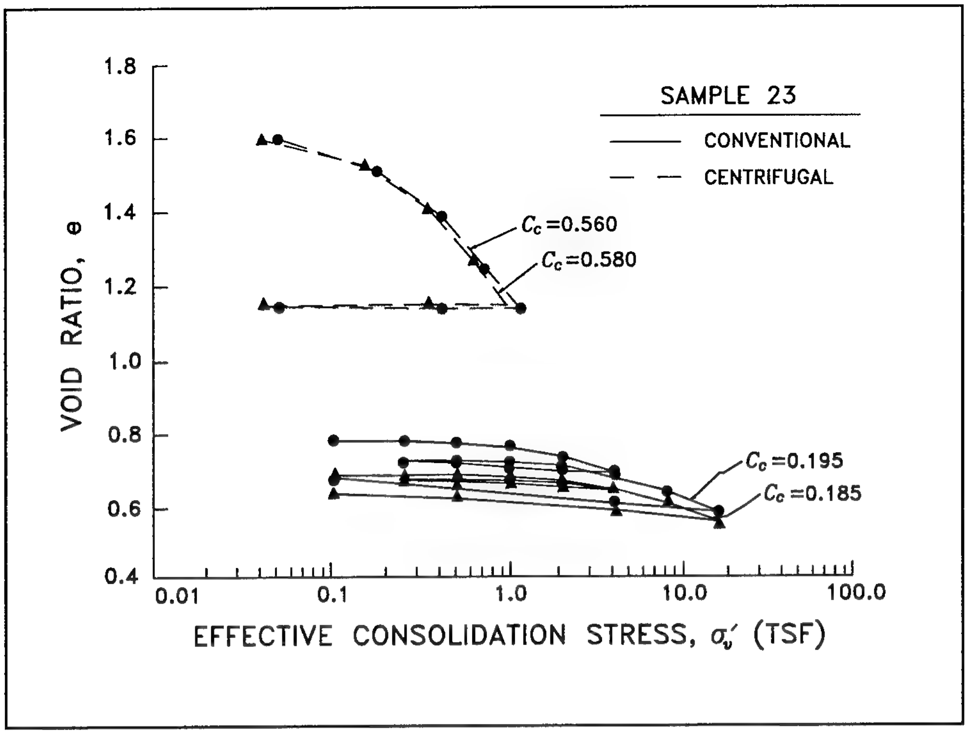

Figure 29. Soil sample 23, comparison of conventional (solid line) and centrifugal (dashed line) consolidation tests

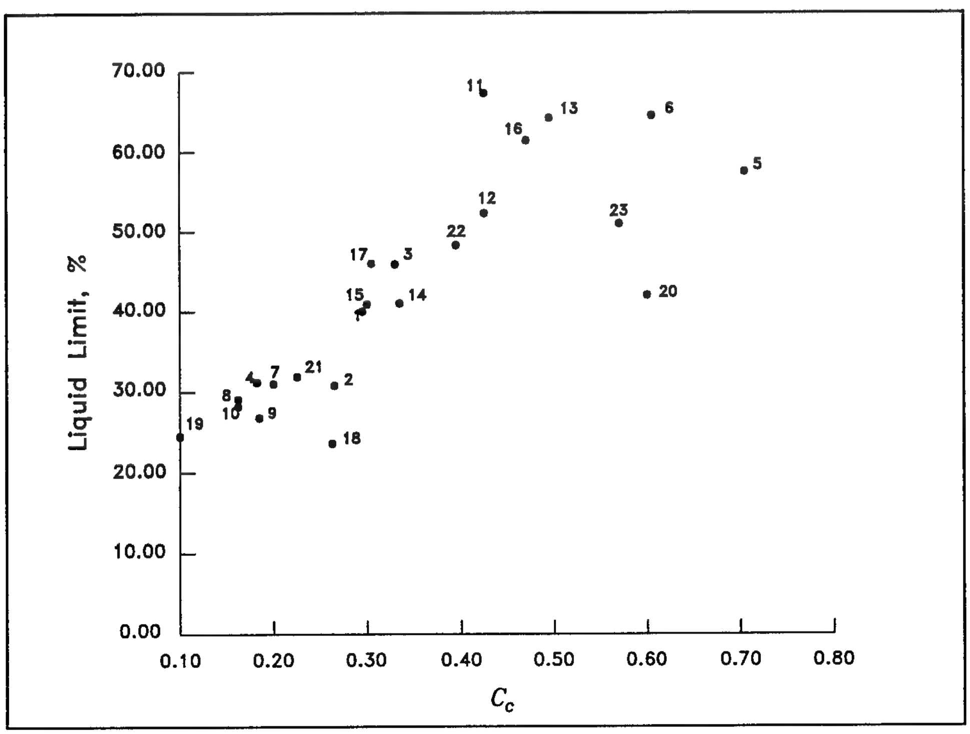

Figure 30. Liquid limit versus C_c, 1-hr spin time

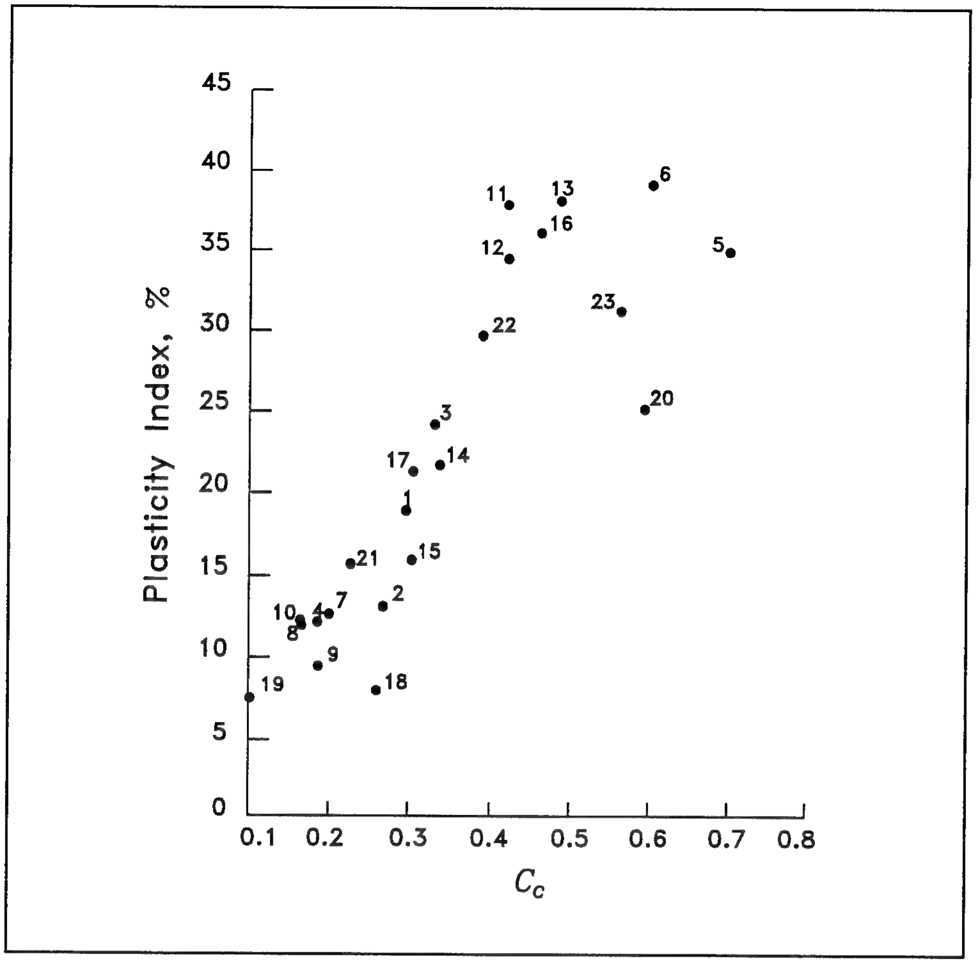

Figure 31. Plasticity index versus C_c, 1-hr spin time

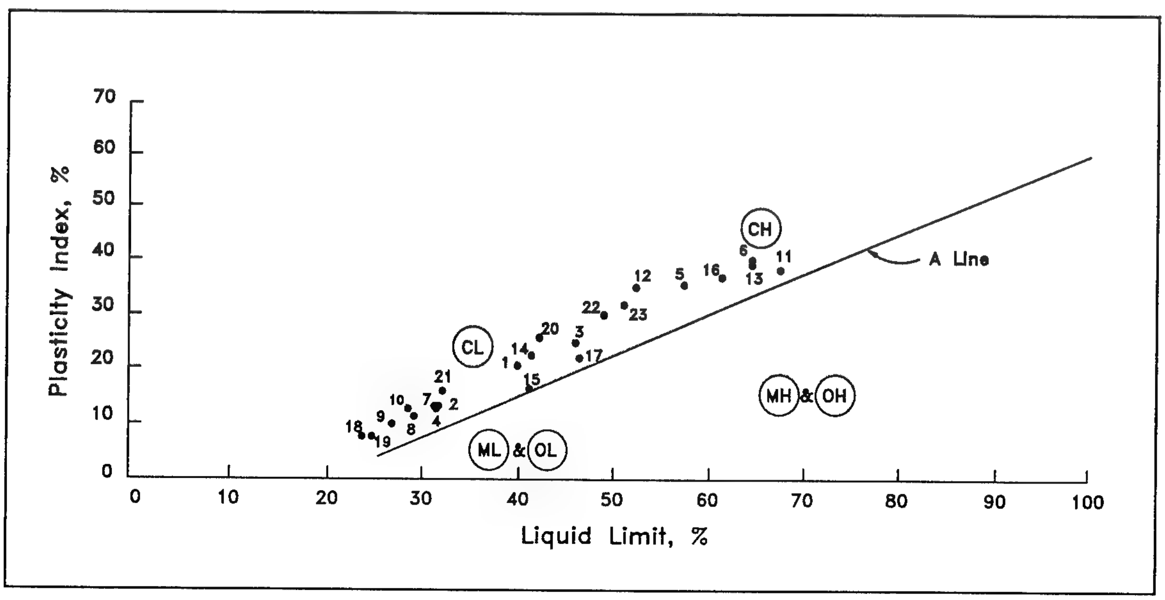

Figure 32. Plasticity chart

be inferred. A potentially more useful interpretation of the information would be values of C_c for ranges of Atterberg limits. For example:

$$LL < 35; PI < 15 \Rightarrow 0.10 < C_c < 0.27$$

$$35 < LL < 50; 15 < PI < 30 \Rightarrow 0.28 < C_c < 0.40$$

$$LL > 50; PI > 30 \Rightarrow 0.41 < C_c < 0.71$$

This usefulness is obviously limited by the correlation between centrifugal and conventional consolidation test results. A plausible argument can be made for the centrifuge test results since they could be considered a better representation of the virgin compression, i.e., disturbance effects are not present.

A good correlation was observed between void ratio at 1 tsf and C_c from the centrifuge consolidation tests (Figure 33). The data points not "fitting" the relationship were common among various plots, i.e., Figures 30-32. Specifically, the common points, or soil samples not correlating well, were Nos. 5, 6, 11, 18, 20, and 23. More important, comparison of Figures 32 and 33 shows that the proximity of a datum to another on Figure 32 holds for Figure 33. Samples having low plasticity characteristics on Figure 32 also have low e and C_c on Figure 33, whereas highly plastic soils on Figure 32 have high values of C_c and e on Figure 33. Thus, the placement of a datum on Figure 32 could be roughly mapped to Figure 33 and vice versa. Samples 15, 11, 18, 20, and 23 appear as "outliers" on plots, the plasticity chart shown in Figures 30, 31, and 33, but not in Figure 32.

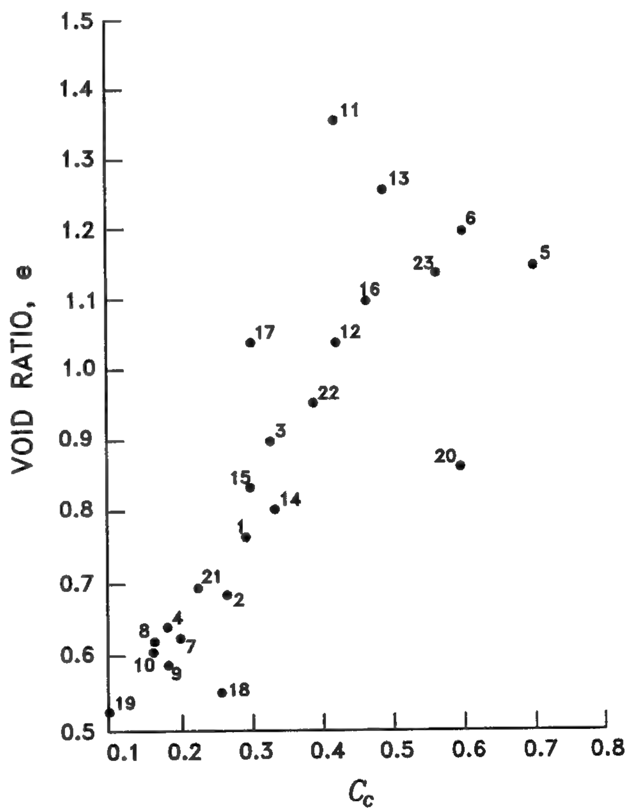

Figure 33. Void ratio at 1 tsf versus C_c, 1-hr spin time

Samples 5, 11, 18, 20, and 23 appear as outliers on plots shown in Figures 30 and 31. The grouping of samples appears better in Figure 33, but samples 5, 11, 18, and 20 still fall outside the group. None of the points appear to be exceptional on the plasticity chart (Figure 32). The physicochemical properties in Table 3 were investigated to determine if a possible explanation for

Table 3
Physicochemical Properties of Soil Samples

Sample No.	pH	Total Soluble Salts, ppm	Exchangeable Cations, ppm			Sodium Adsorption Ratio	Sodium percent
			Na	Ca	Mg		
1	7.6	3,564	31	423	48	0	0
2	7.6	1,404	30	153	42	1	0
3	8.0	2,669	188	213	87	3	2
4	7.4	873	14	75	21	0	0
5	7.8	885	171	19	7	9	10
6	8.2	2,125	187	142	62	3	3
7	7.5	592	11	47	12	0	0
8	7.6	677	56	53	25	2	1
9	7.5	624	110	18	9	5	6
10	6.8	331	17	19	9	1	0
11	8.0	1,356	37	156	22	1	0
12	8.0	2,154	448	58	19	13	15
13	7.9	2,463	263	198	20	5	5
14	7.8	7,306	86	1,002	86	1	0
15	7.9	9,603	83	865	366	1	0
16	8.3	3,524	267	240	62	4	4
17	5.1	517	23	22	12	1	0
18	8.3	1,101	296	7	2	25	25
19	8.3	2,277	289	58	42	9	11
20	7.5	867	29	96	15	1	0
21	7.9	1,505	73	159	40	1	0
22	8.0	1,113	240	25	7	11	12
23	8.3	867	188	27	4	9	10

the outliers could be found. There appears to be no remarkable set of properties common to these samples that explains falling outside the group trend. While it is possible that inconsistency in testing technique may have caused error, this does not explain a consistent outlier. Sample 18 was tested under a number of conditions (see Figures 5, 6, and 14) with relatively repeatable results. Evidently, there are factors affecting the mechanical response that do not affect the Atterberg limits.

4 Conclusions

The following conclusions may be drawn concerning the results of the centrifugal consolidation testing program:

a. The centrifuge consolidation test provides an alternative procedure to classify cohesive soils. Because the test is based on the intrinsic consolidation properties of the soil, parameters measured in the centrifuge consolidation test are more directly related to engineering behavior than the Atterberg limit test.

b. Empirical relationships between consolidation parameters obtained from the consolidation test and Atterberg limits are similar to those obtained using other types of consolidation tests on resedimented soils. The similarity in empirical correlation strongly implies that the centrifuge test gives an accurate assessment of virgin consolidation properties.

c. The initial water content of the slurry greatly flattens the consolidation curve. The curve is flatter because the initial water content falls to the left of the virgin compression curve. The lower the initial water content, the flatter the curve will be, and the error in the virgin curve will be greater.

d. More study is needed to determine the most appropriate initial water content. Two criteria must be considered. First, the initial water content must be chosen so that results will be uniform and consistent; otherwise, results obtained will depend too strongly on initial conditions. Second, the best approximation of the virgin curve is obtained from samples having the highest initial water content. It is proposed that both criteria are met by testing from an as-sedimented state (i.e., under self-weight loading only) in which the soil is mixed to its least dense consistency as for a self-weight consolidation test. The equilibrium water content in the as-sedimented state should be a function of plasticity and thus may provide additional information for classification purposes. Consistency would result from the fact that the procedure for obtaining the as-sedimented state would not depend on prior knowledge of the Atterberg limits.

e. Difficulties in comparing consolidation tests for undisturbed specimens to the virgin curves obtained in the centrifuge test are the result of a number of factors. The major factor is the low initial water content of the undisturbed specimen relative to the virgin consolidation curve. Another factor is the influence of stress history and sampling disturbance on the behavior of the undisturbed specimen. For example, in the SHANSEP procedure of Ladd and Foote (1974), it is recommended that specimens need to be loaded as much as two log cycles (on the load scale) beyond the apparent preconsolidation stress before normally consolidated behavior is reached. In most cases, two log cycles of loading extend well beyond the range (~ 1 tsf) for which the centrifuge test appears to be practical. Beyond the 1-tsf range, the compression curve flattens, and thus, undisturbed specimens converge to a virgin curve different than that measured in the centrifuge test. The effect of initial conditions on the consolidation is uncertain. The comparison between undisturbed specimens and the centrifuge test is not too different from that between the reconstituted specimen in a standard consolidation test and the centrifuge test. Therefore, of all factors possibly influencing consolidation behavior, the initial water content of the specimen appears to be the most important.

f. The centrifuge consolidation test appears to be particularly applicable to dredged materials. As demonstrated, the test provides a means to span the loading range between the self-weight consolidation test and the conventional consolidation test. There is reason to believe that both self-weight and conventional tests could be supplanted with the centrifuge test.

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Appendix A
Void Ratio Versus Effective Consolidation Stress Curves for Centrifuge Consolidation Tests

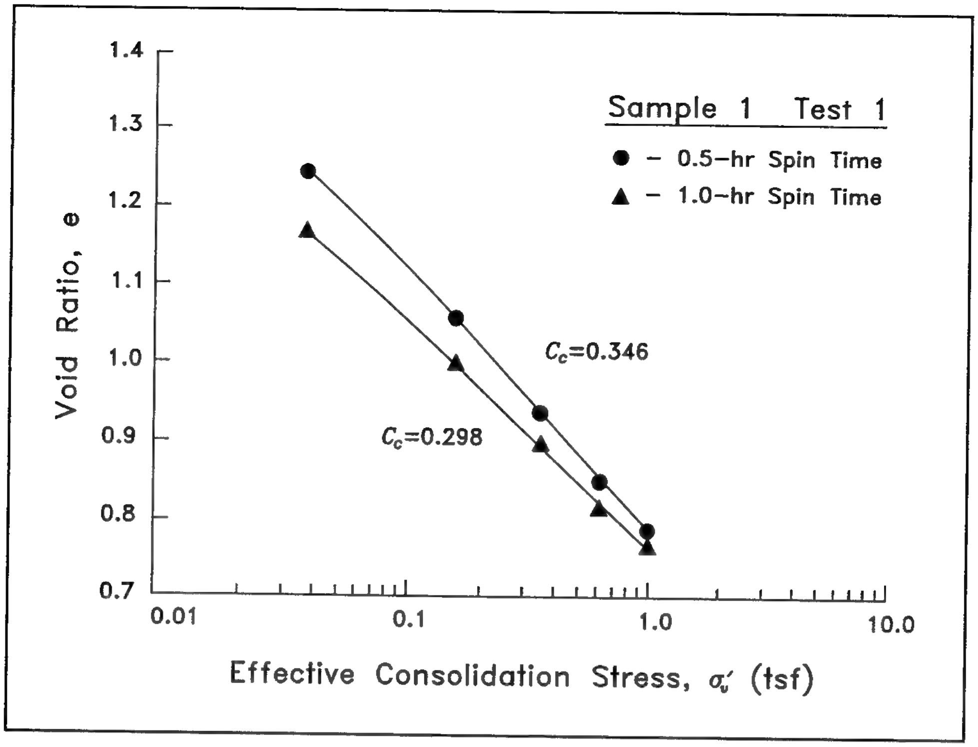


Figure A1. Soil sample 1 Test 1

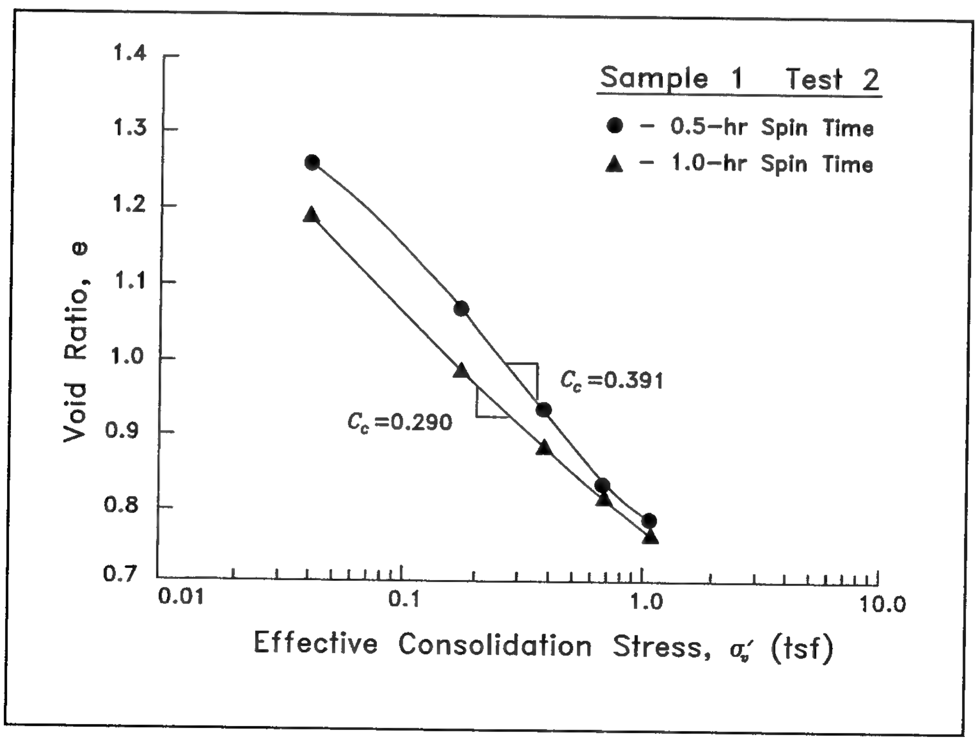


Figure A2. Soil sample 1 Test 2

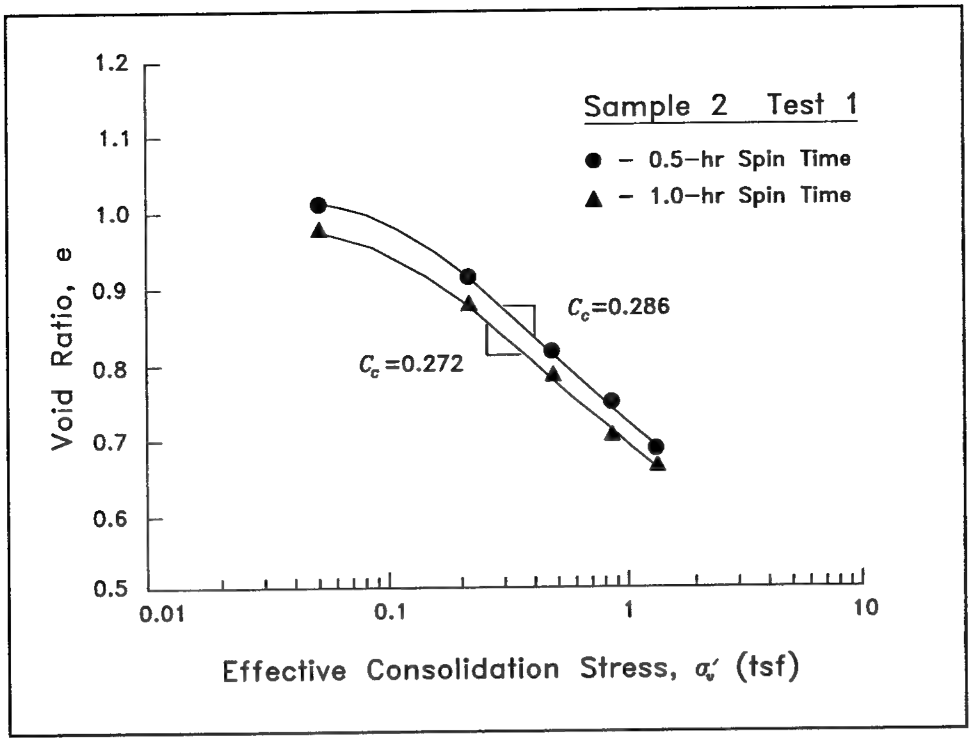


Figure A3. Soil sample 2 Test 1

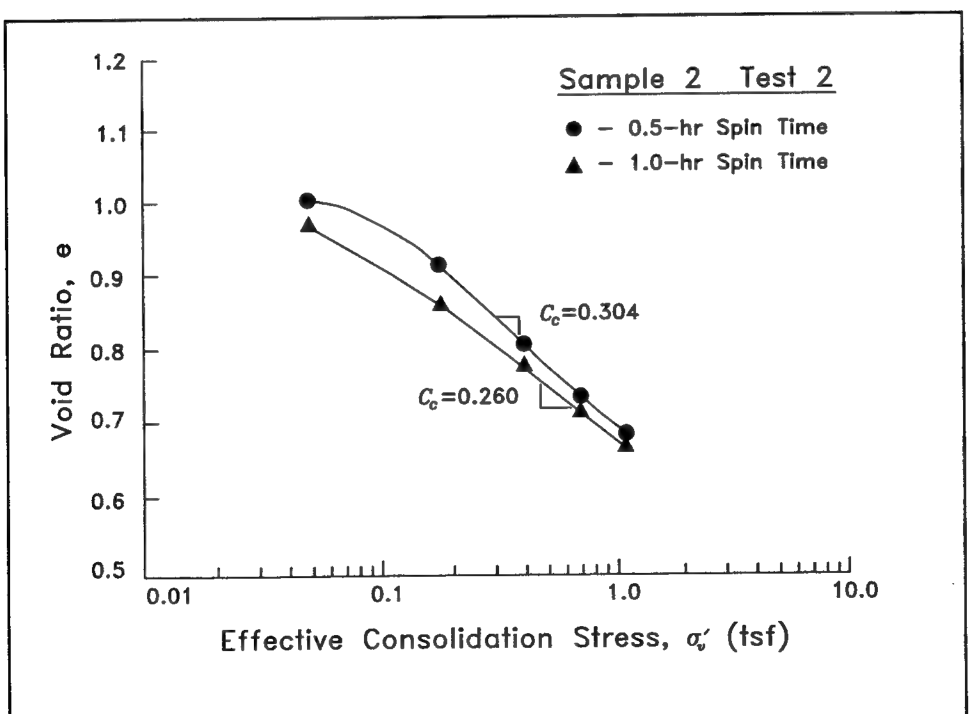


Figure A4. Soil sample 2 Test 2

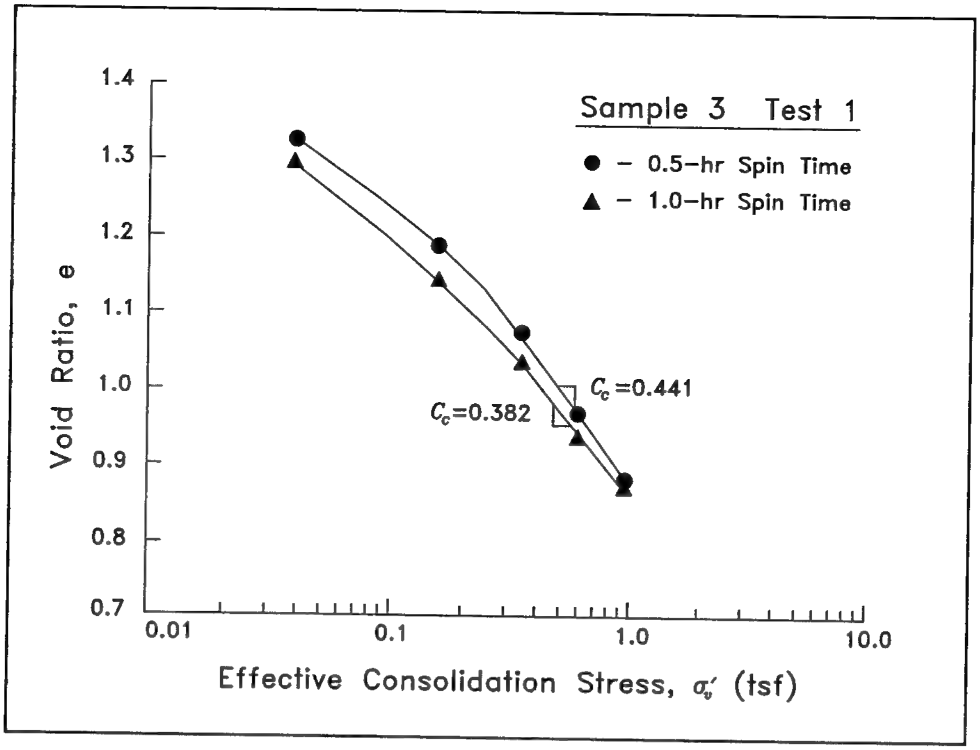


Figure A5. Soil sample 3 Test 1

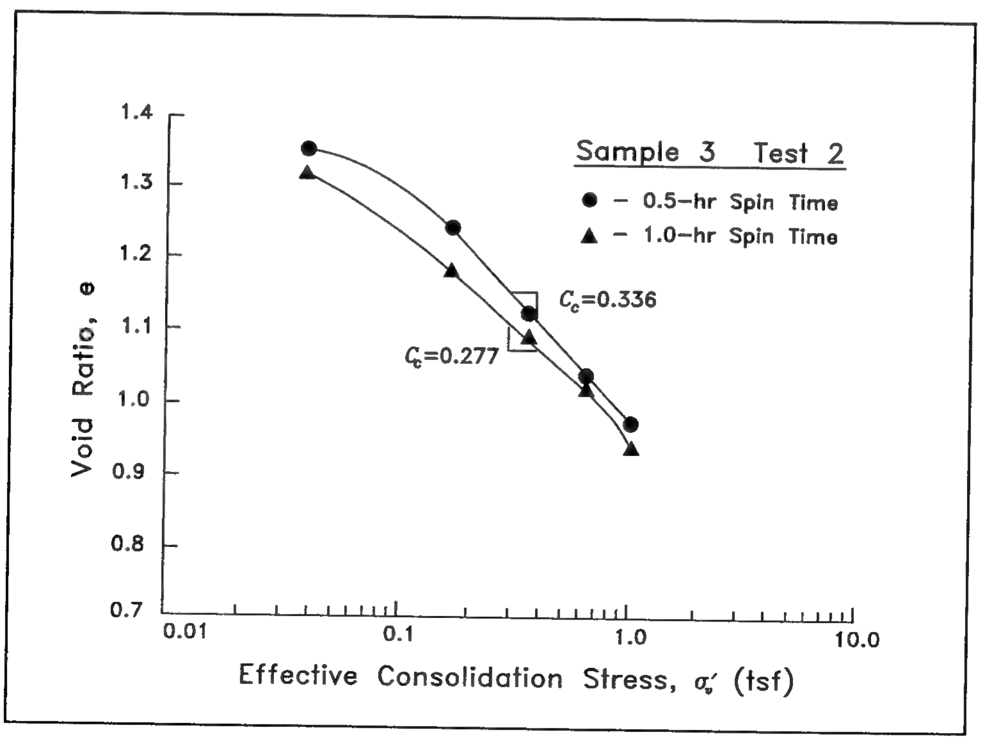


Figure A6. Soil sample 3 Test 2

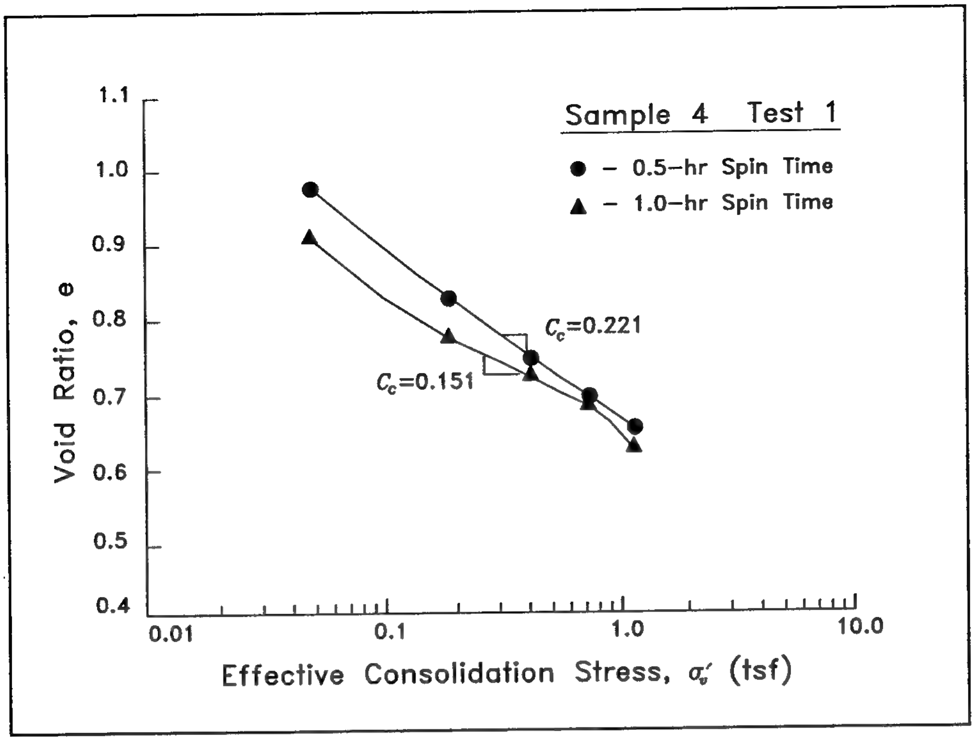


Figure A7. Soil sample 4 Test 1

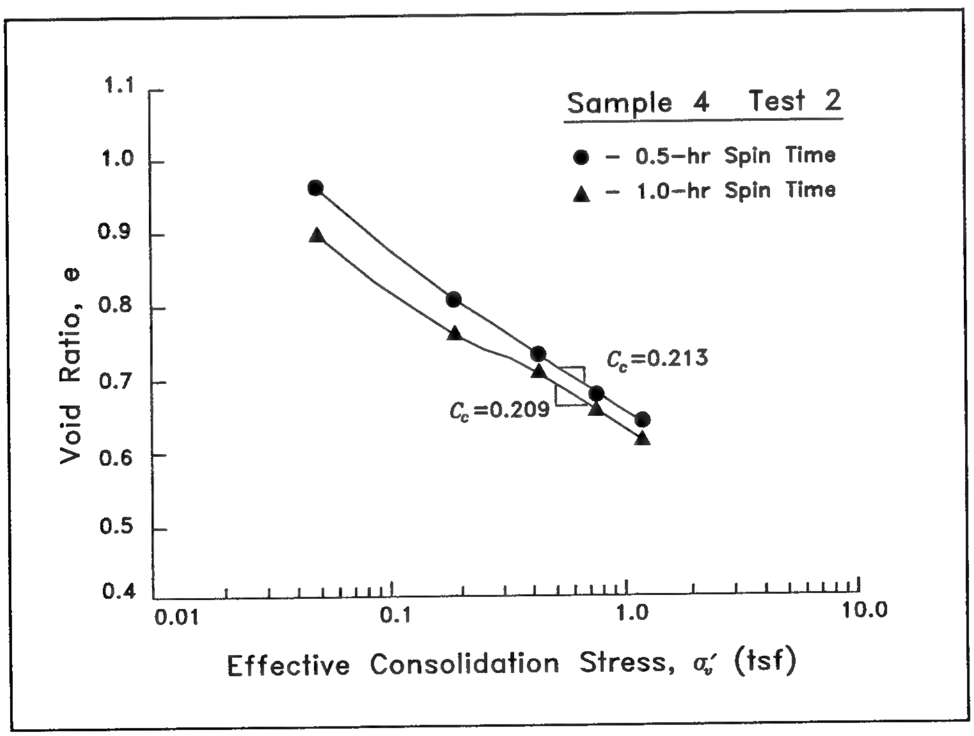


Figure A8. Soil sample 4 Test 2

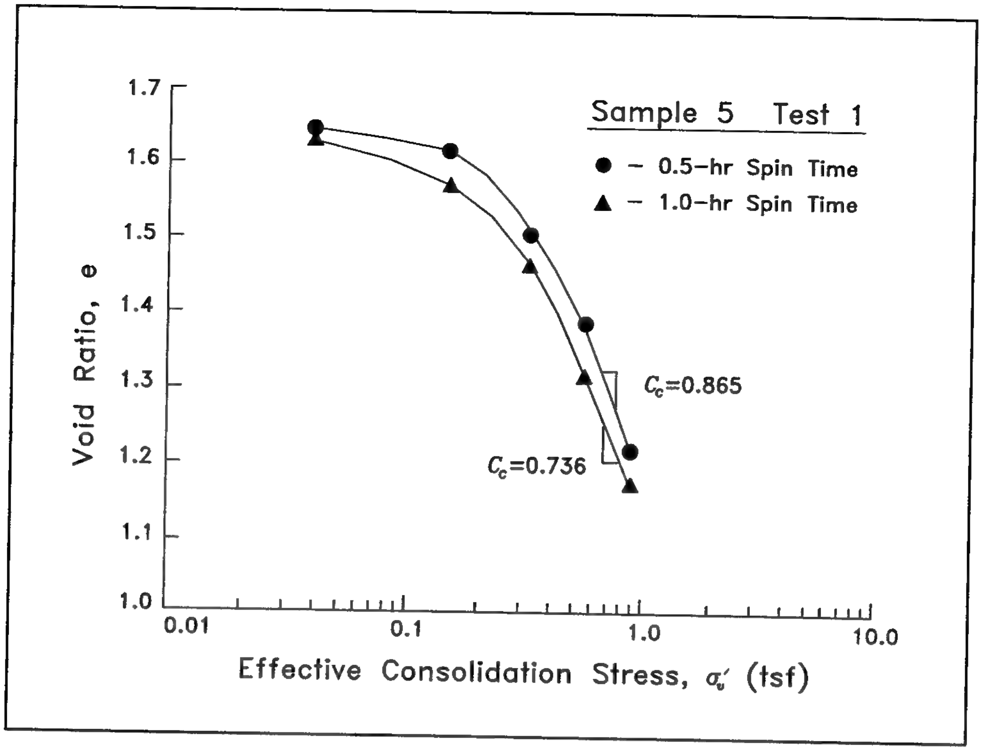


Figure A9. Soil sample 5 Test 1

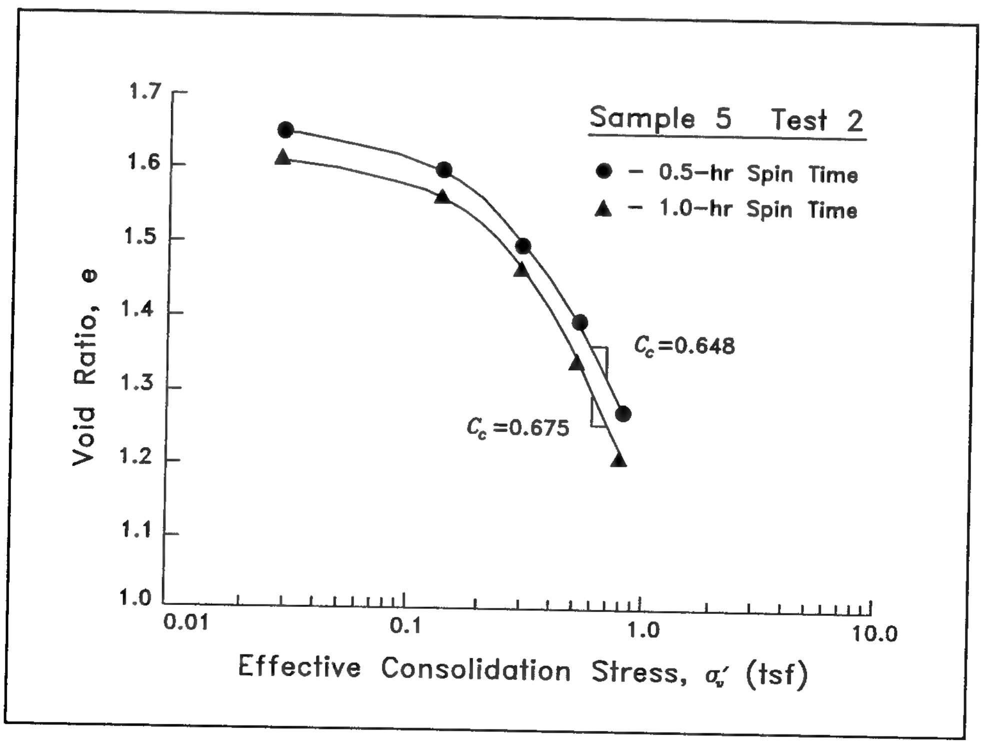


Figure A10. Soil sample 5 Test 2

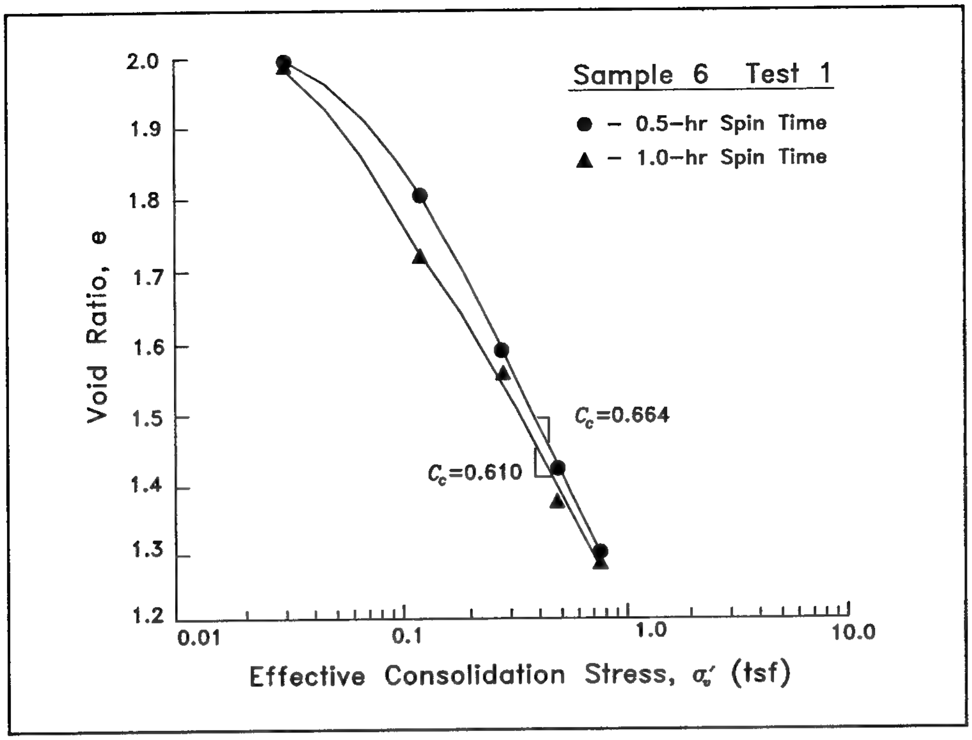


Figure A11. Soil sample 6 Test 1

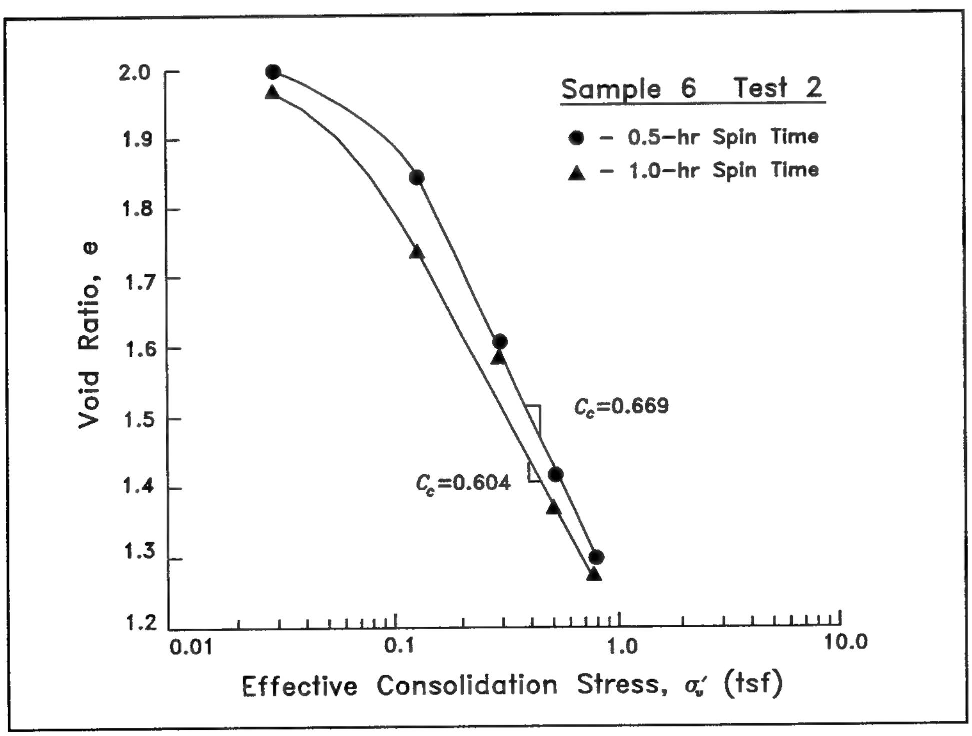
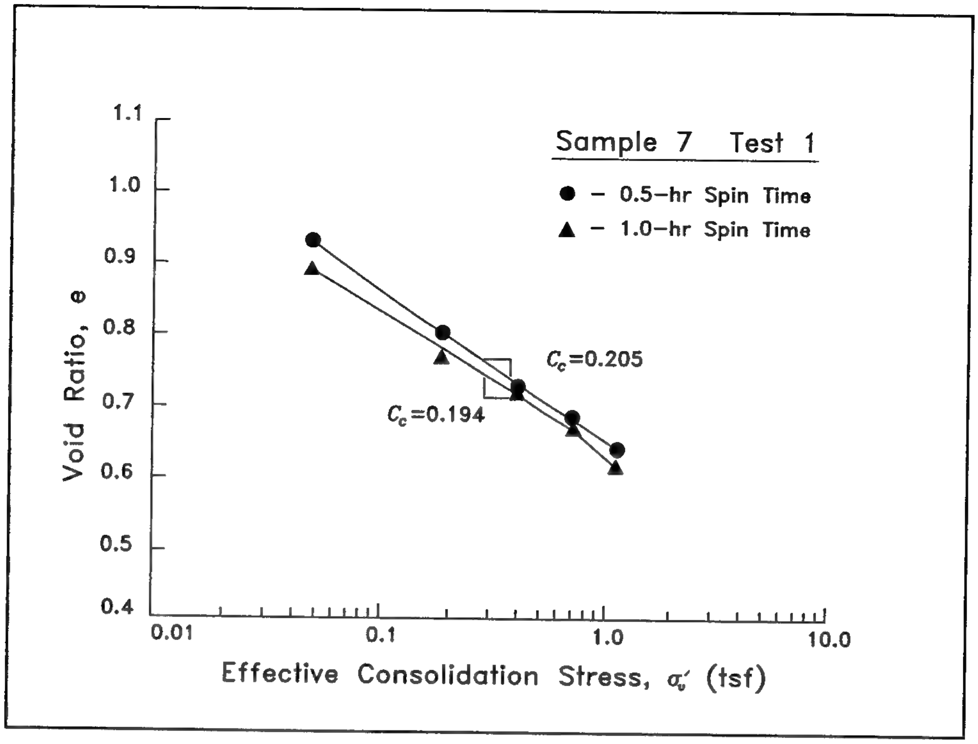


Figure A12. Soil sample 6 Test 2



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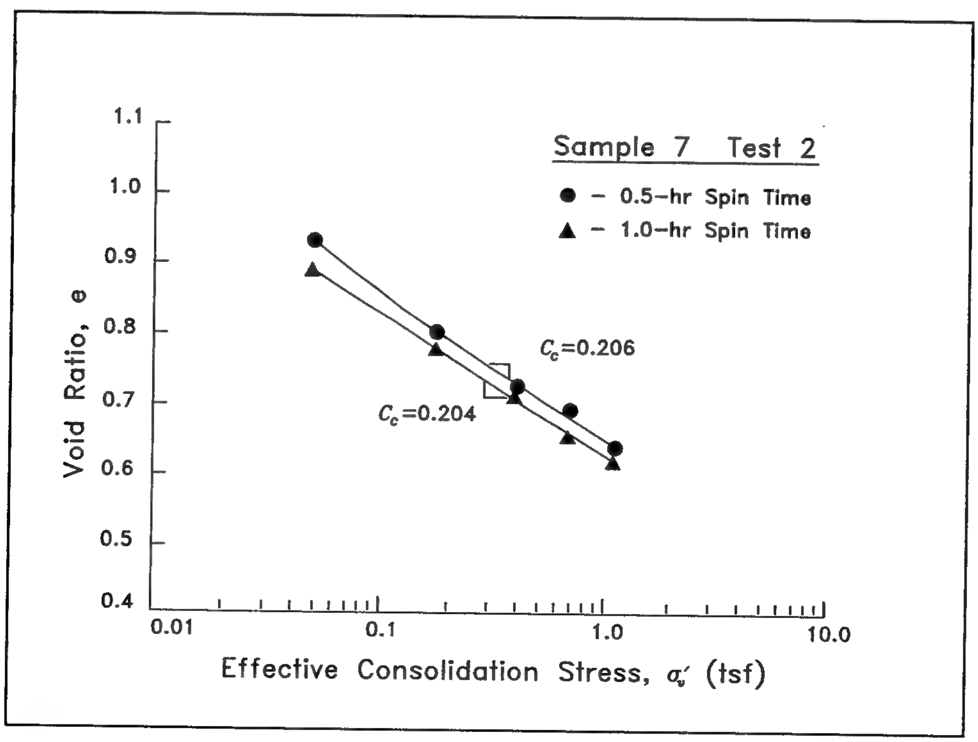

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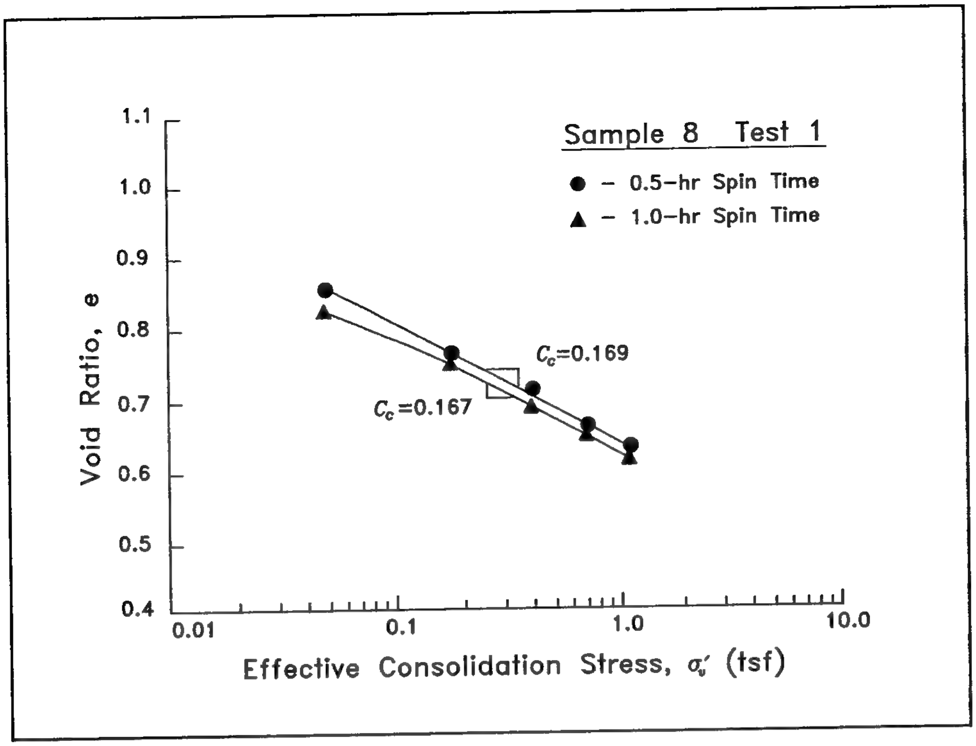

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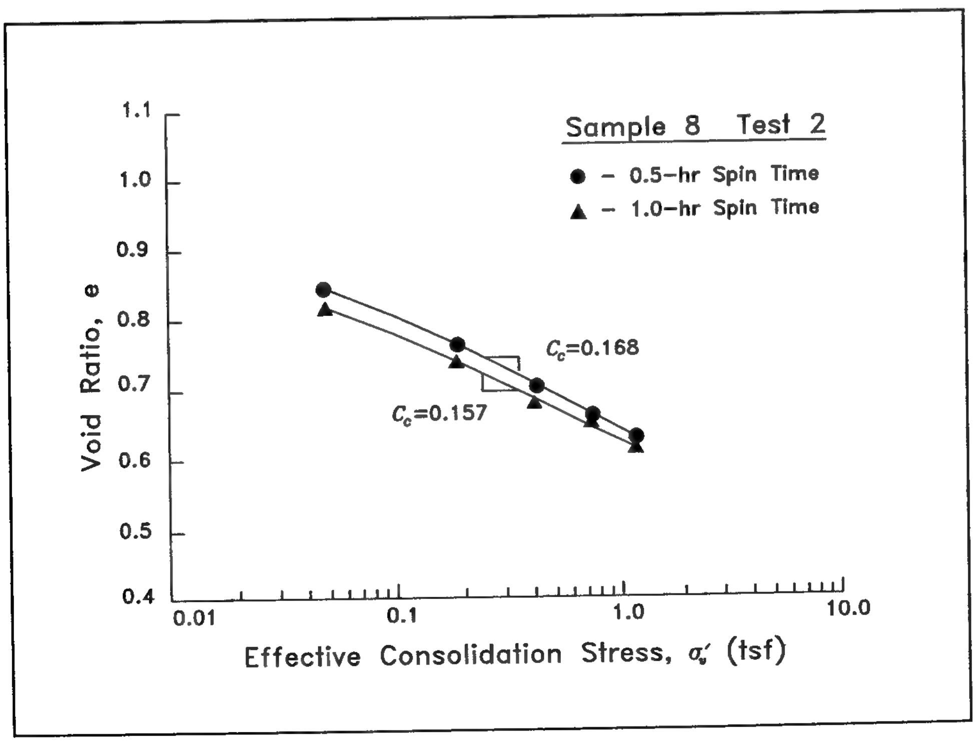

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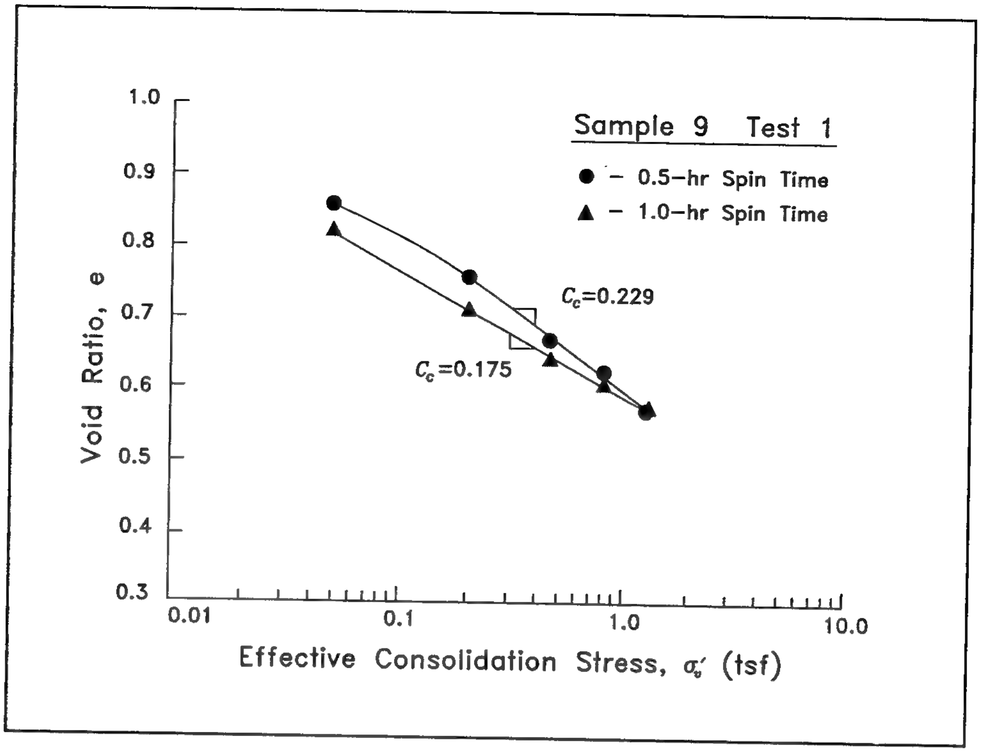

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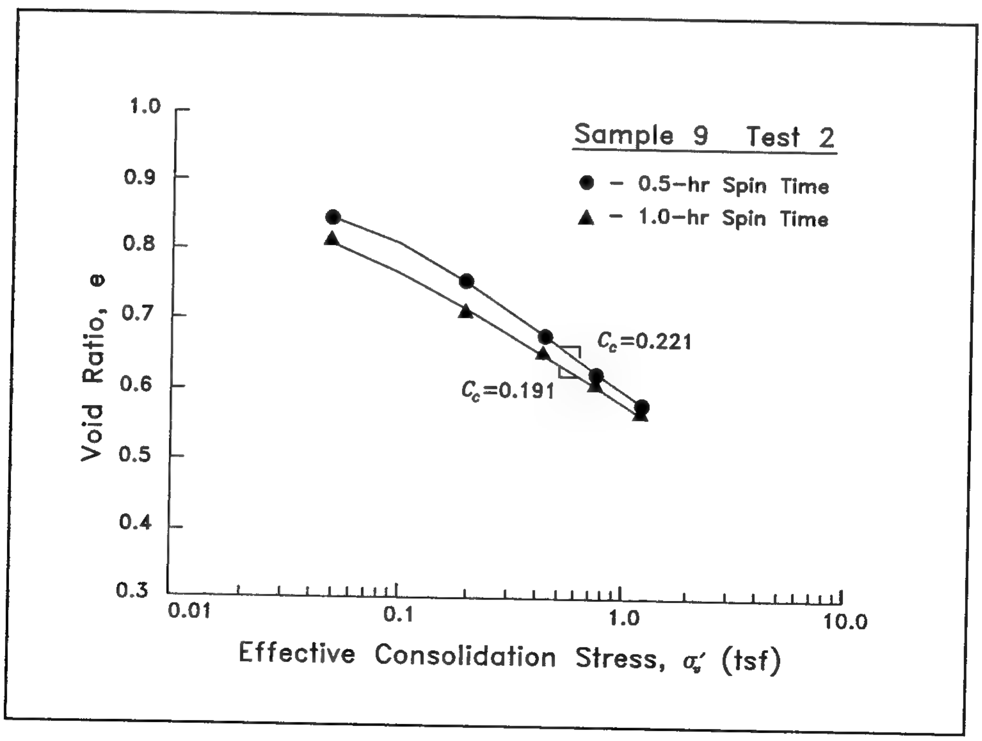

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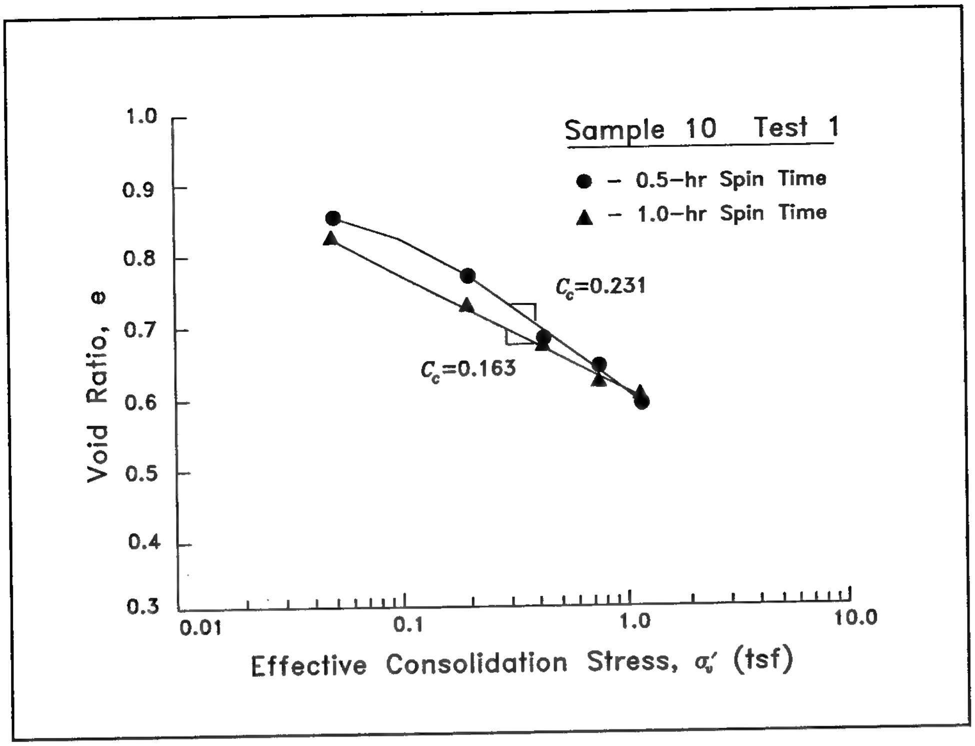

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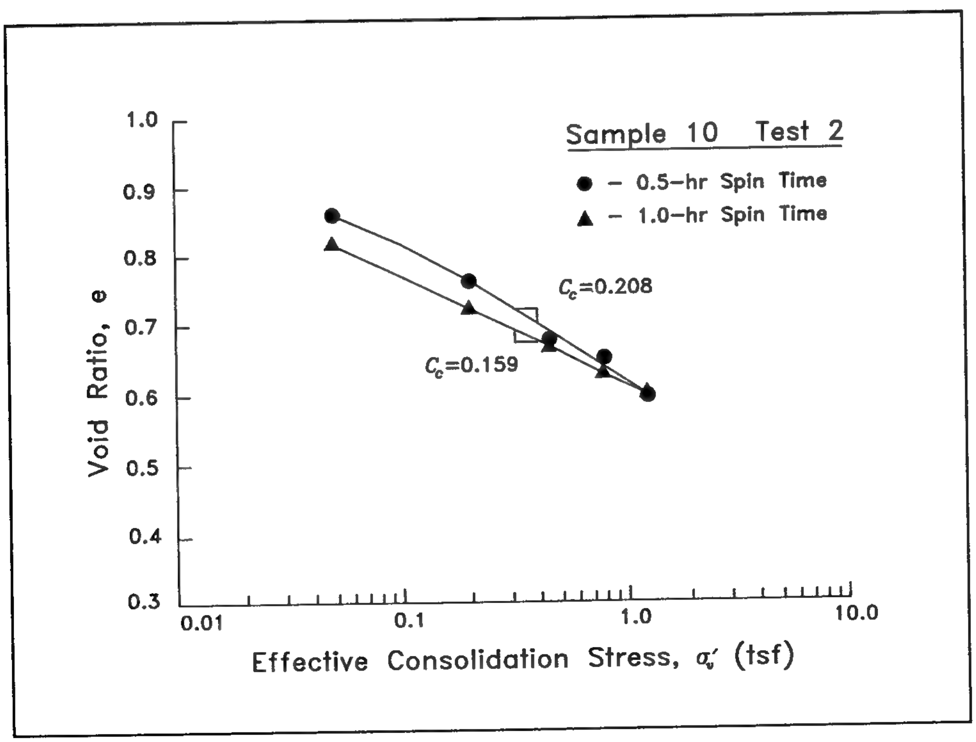

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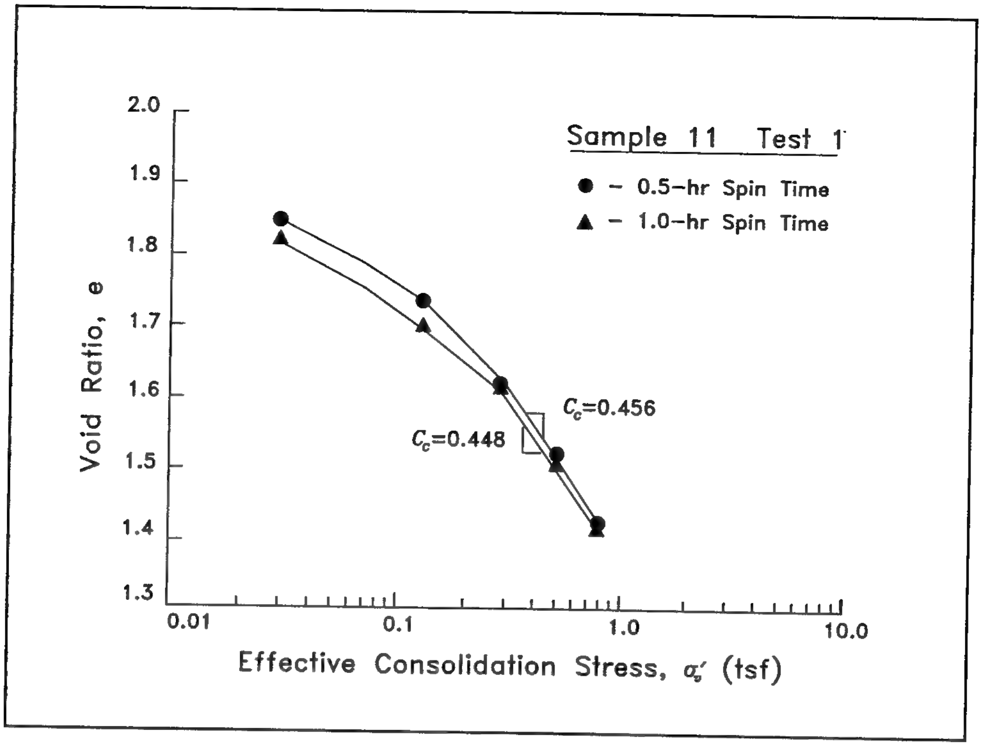

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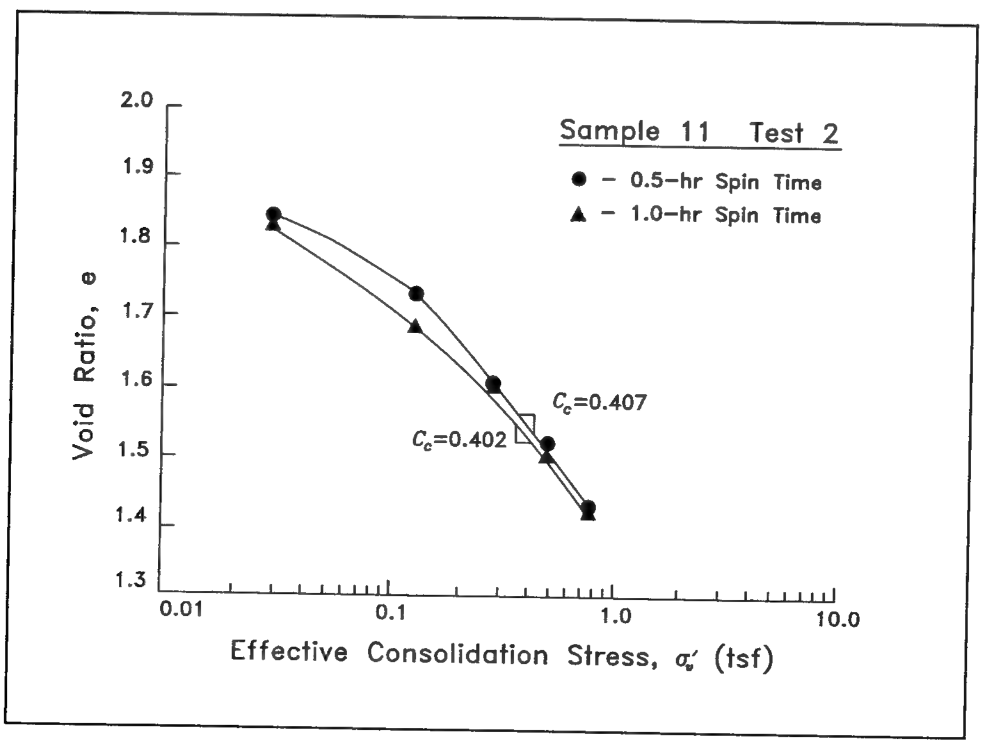
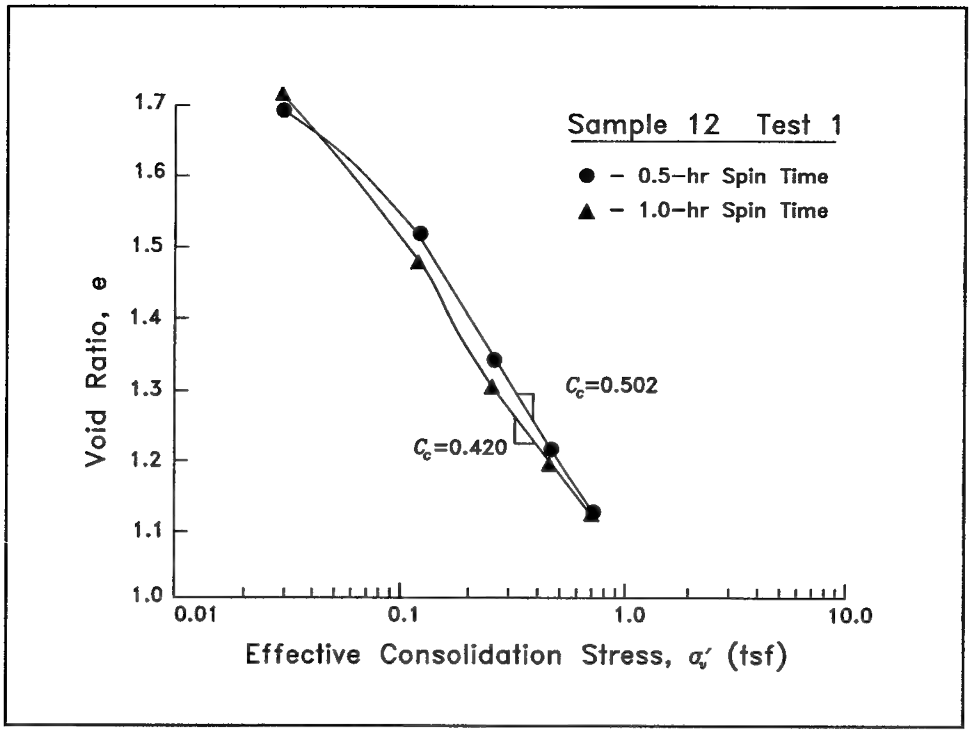


Figure A22. Soil sample 11 test 2



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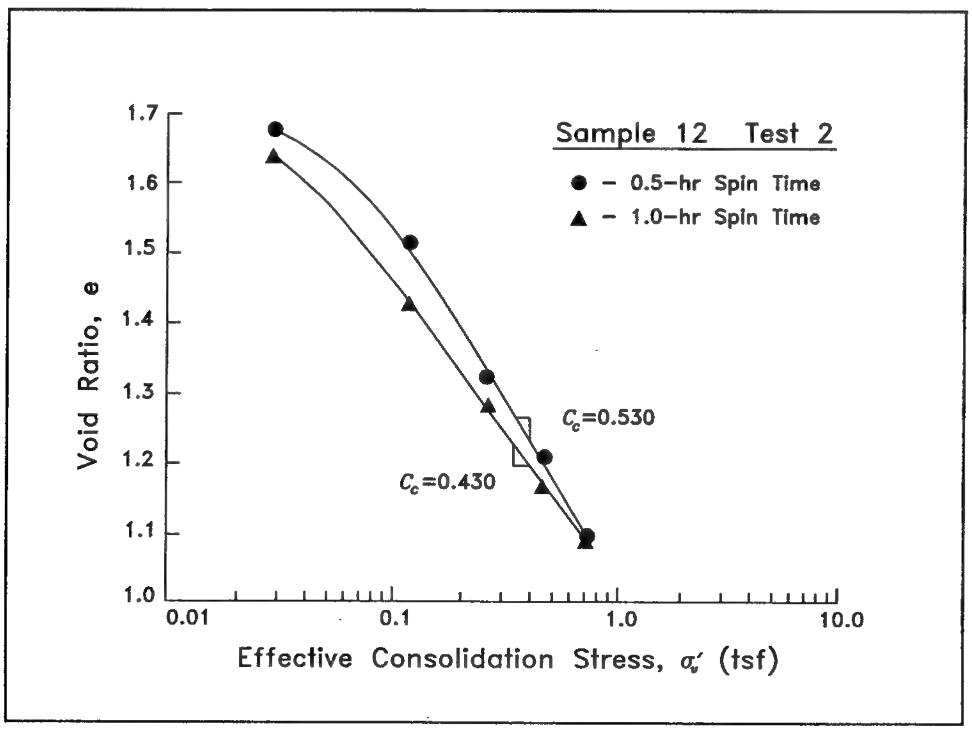

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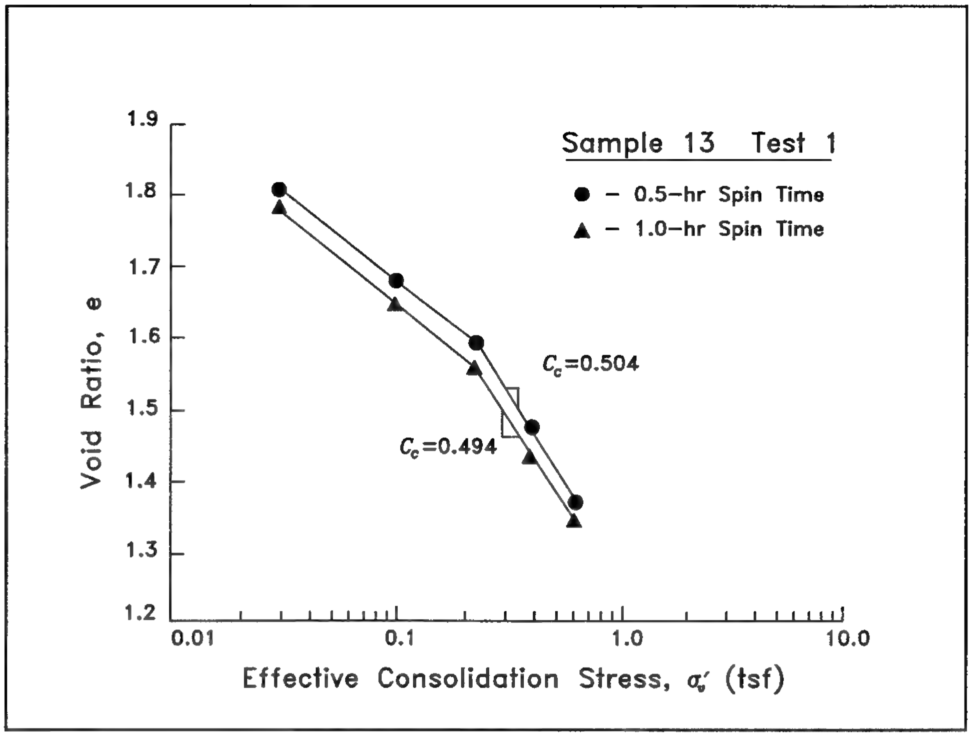

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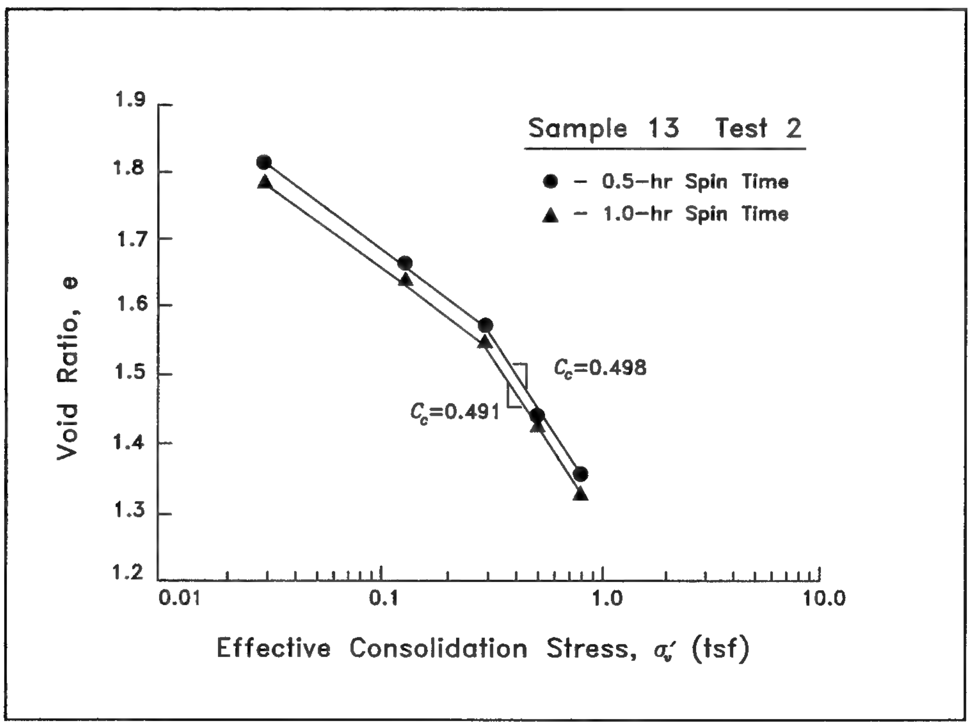
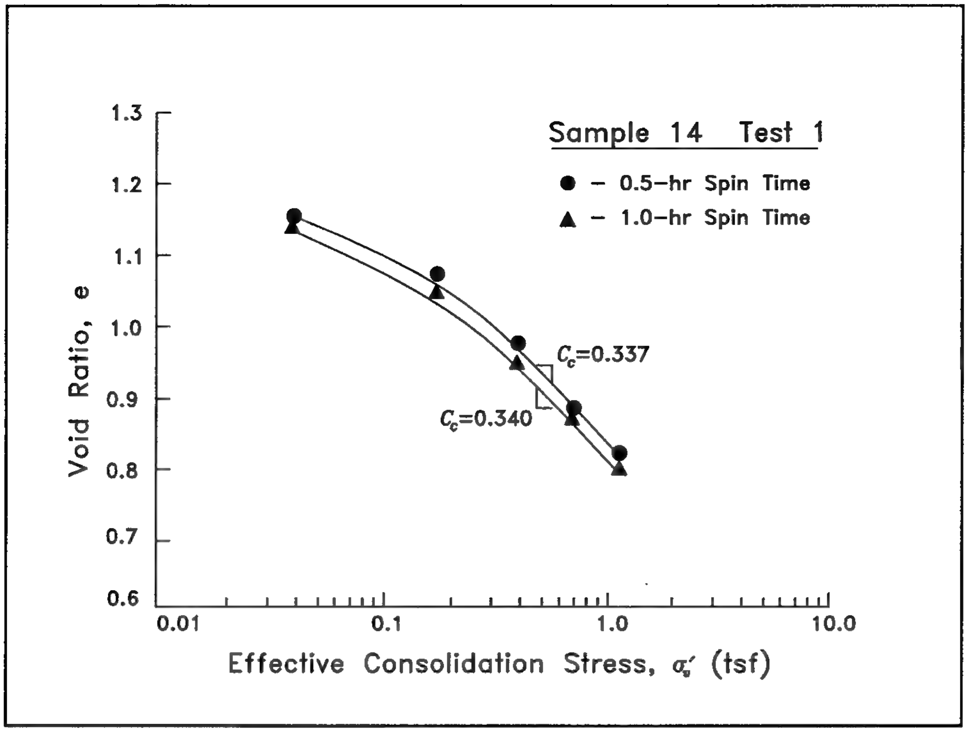


Figure A26. Soil sample 13. Test 2



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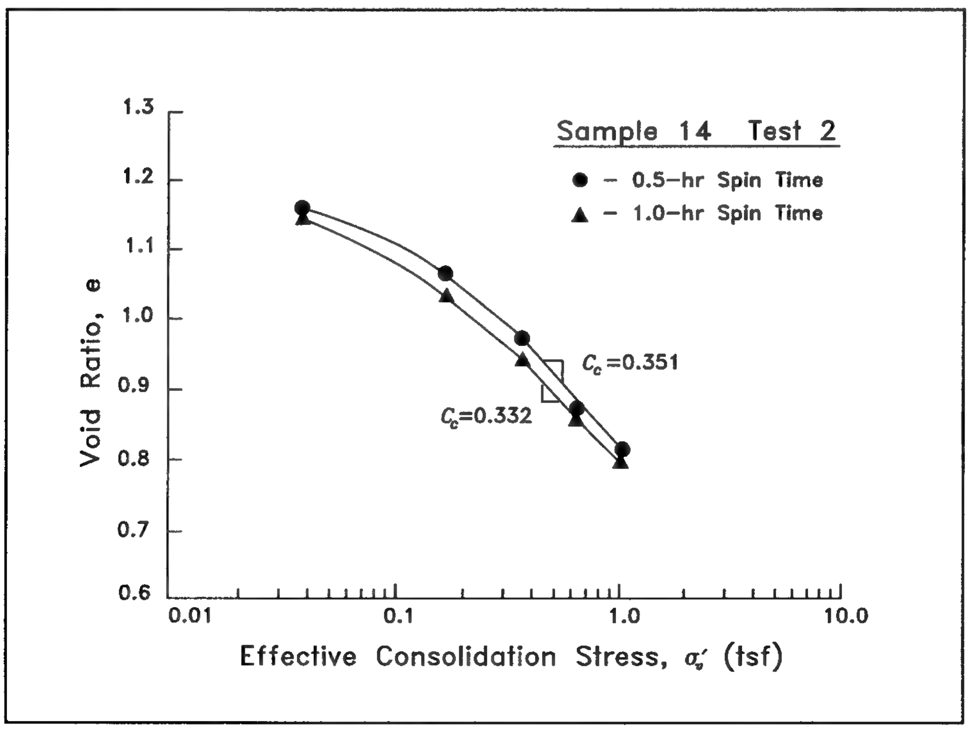

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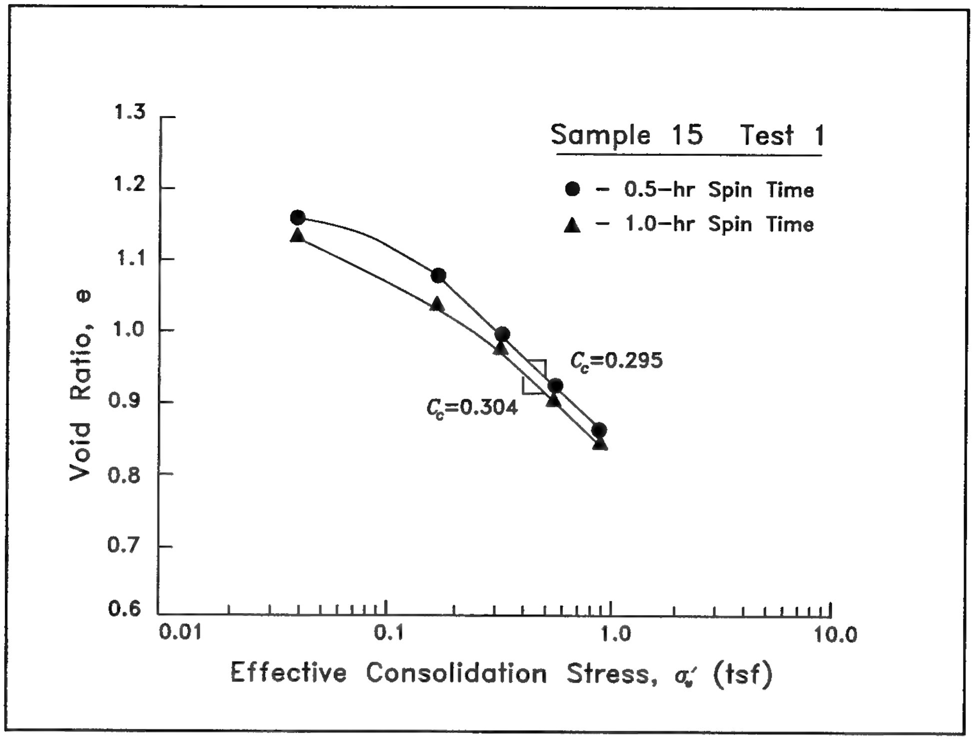
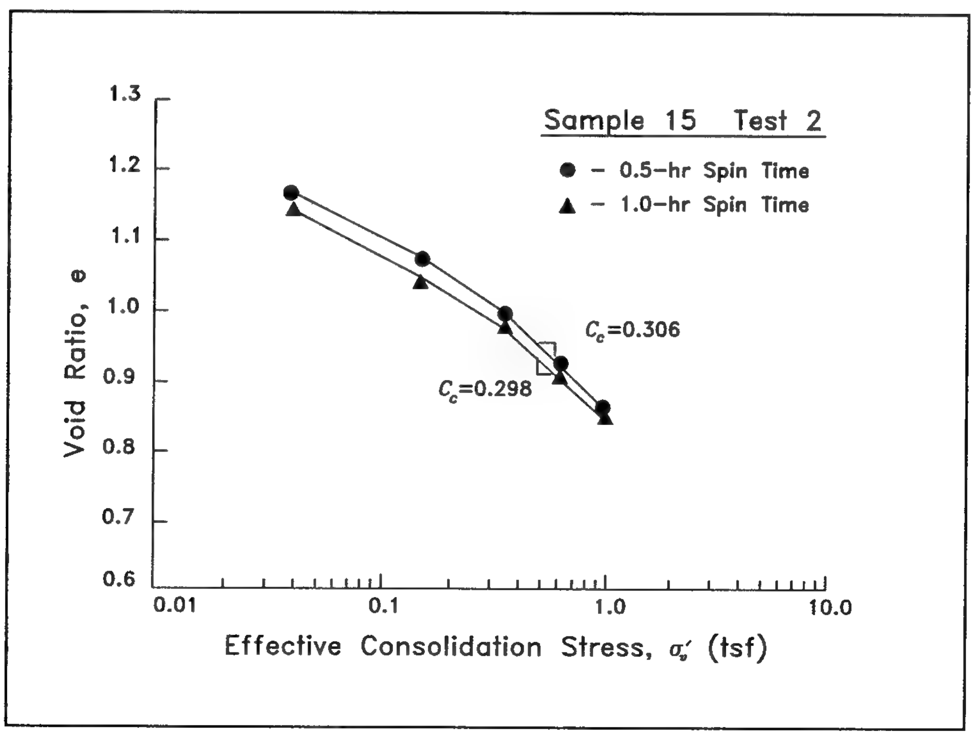


Figure A29. Soil sample 15 Test 1



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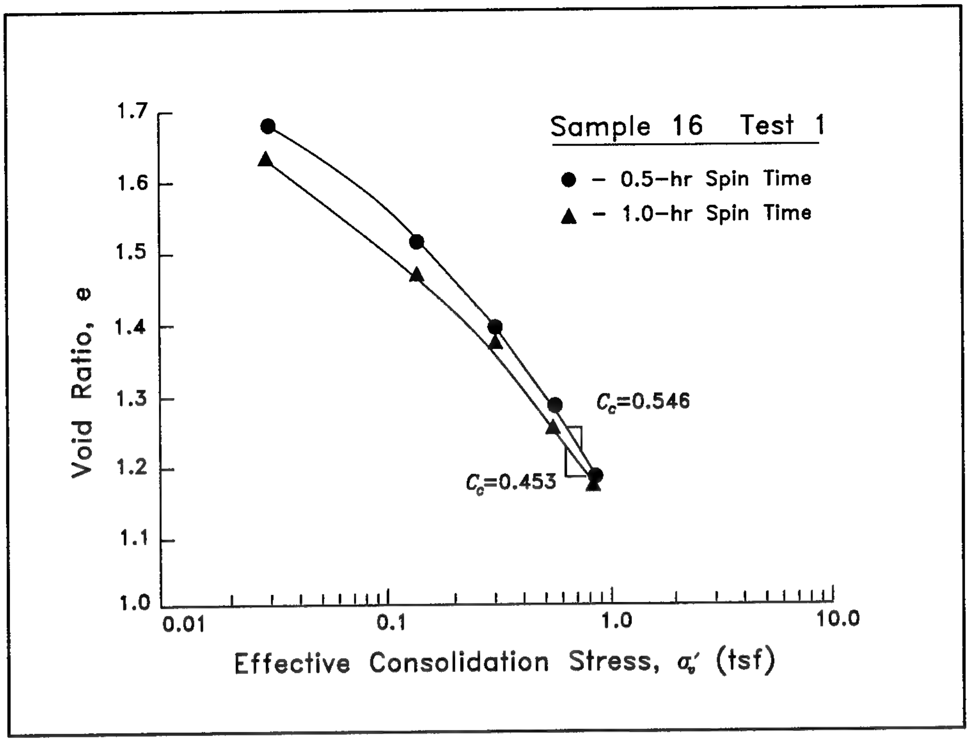

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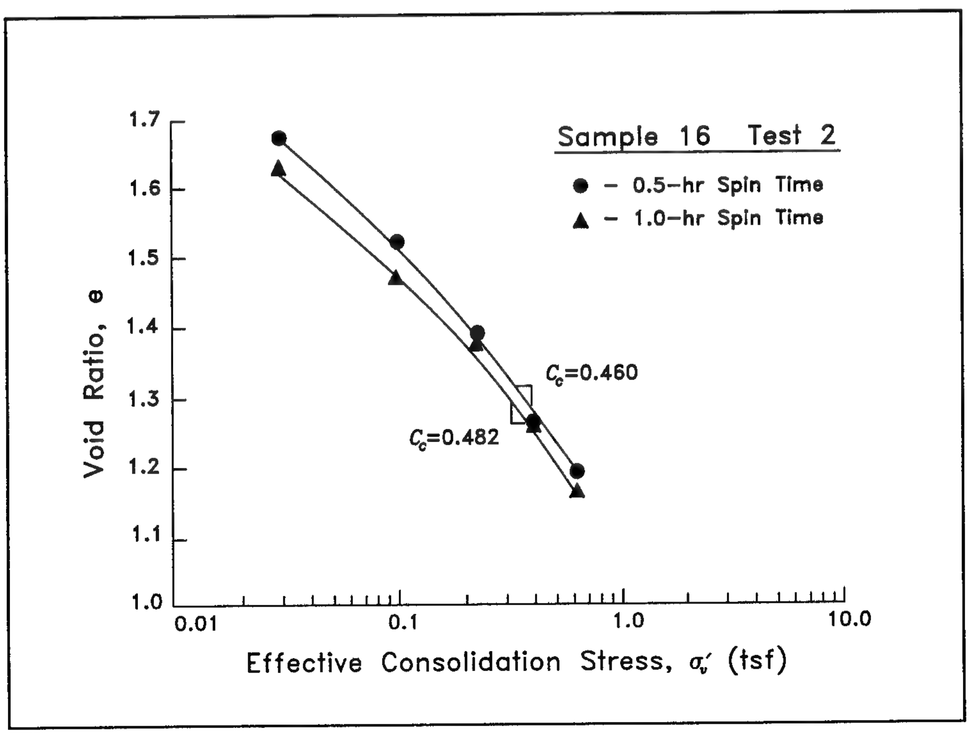

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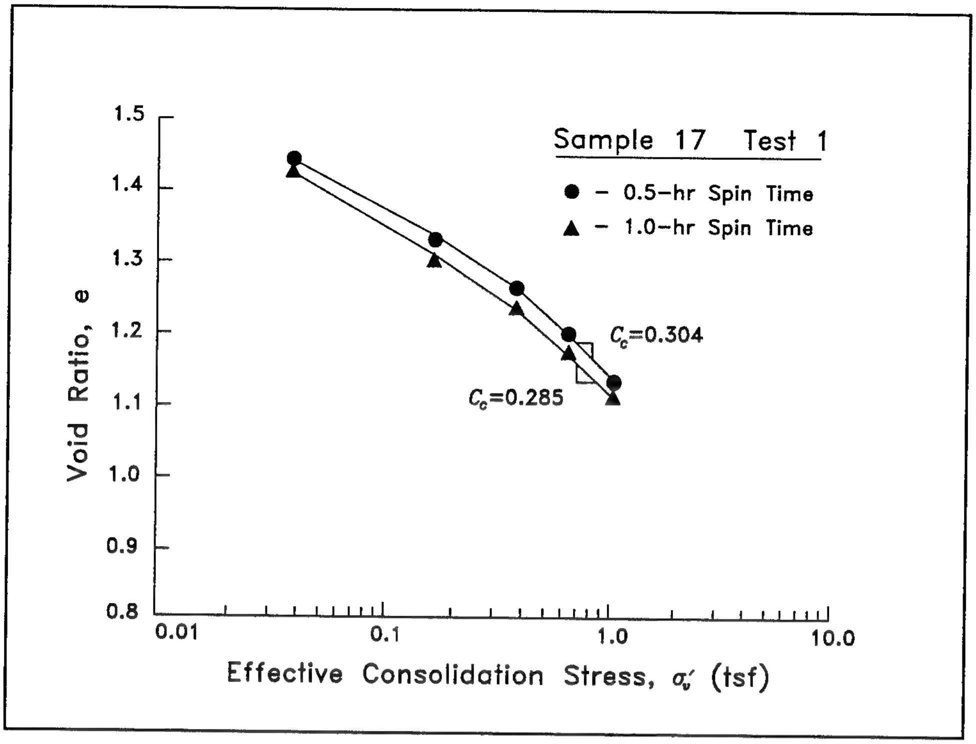

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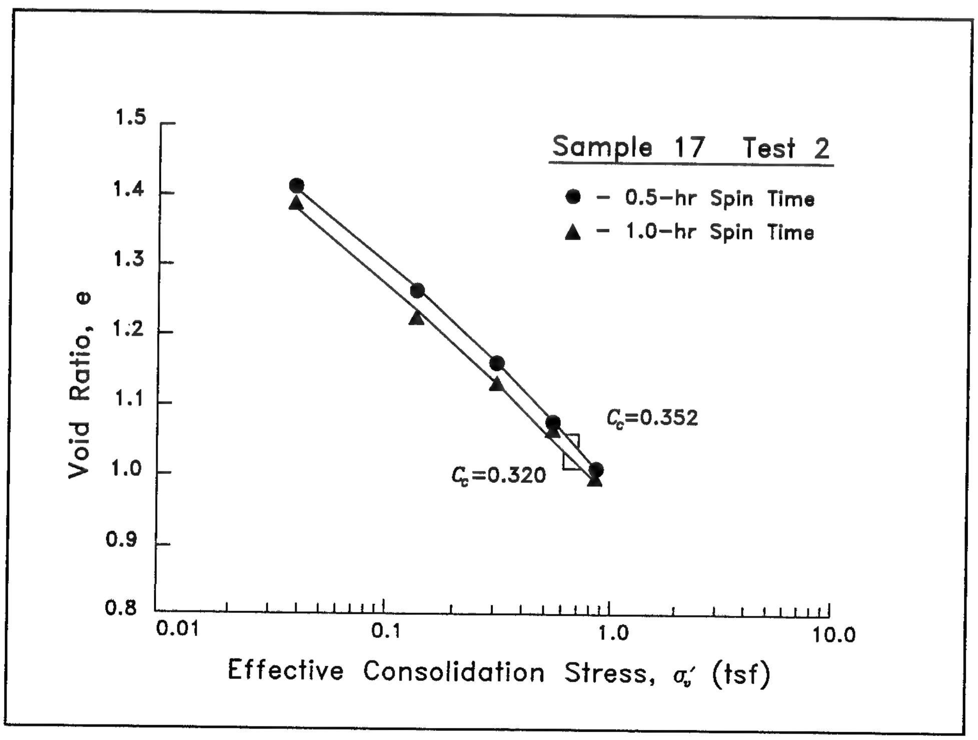

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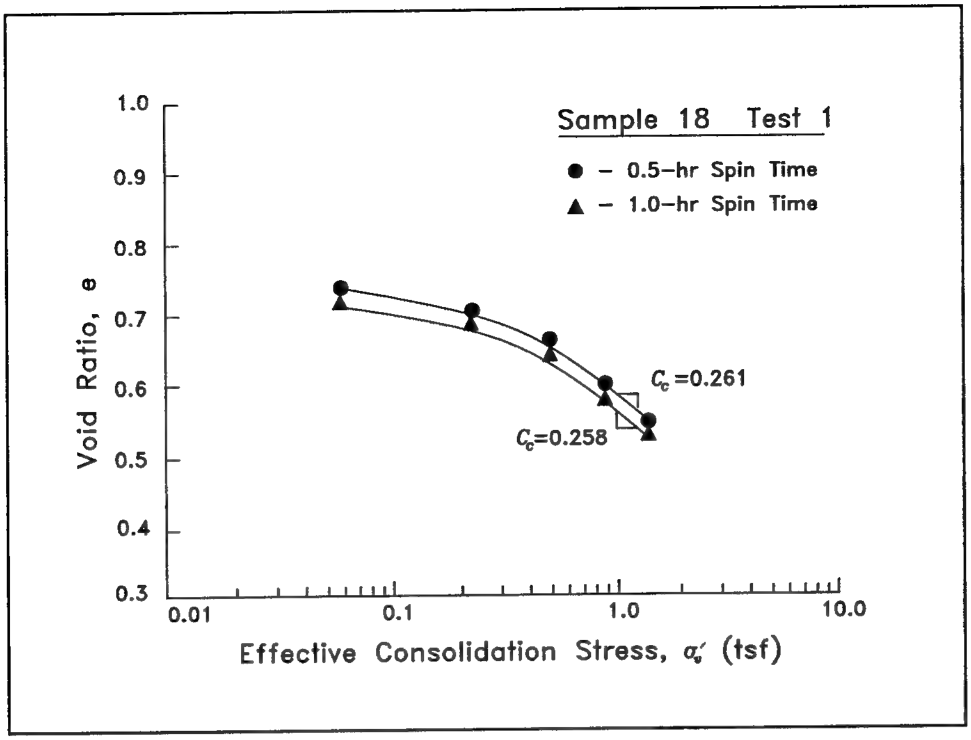

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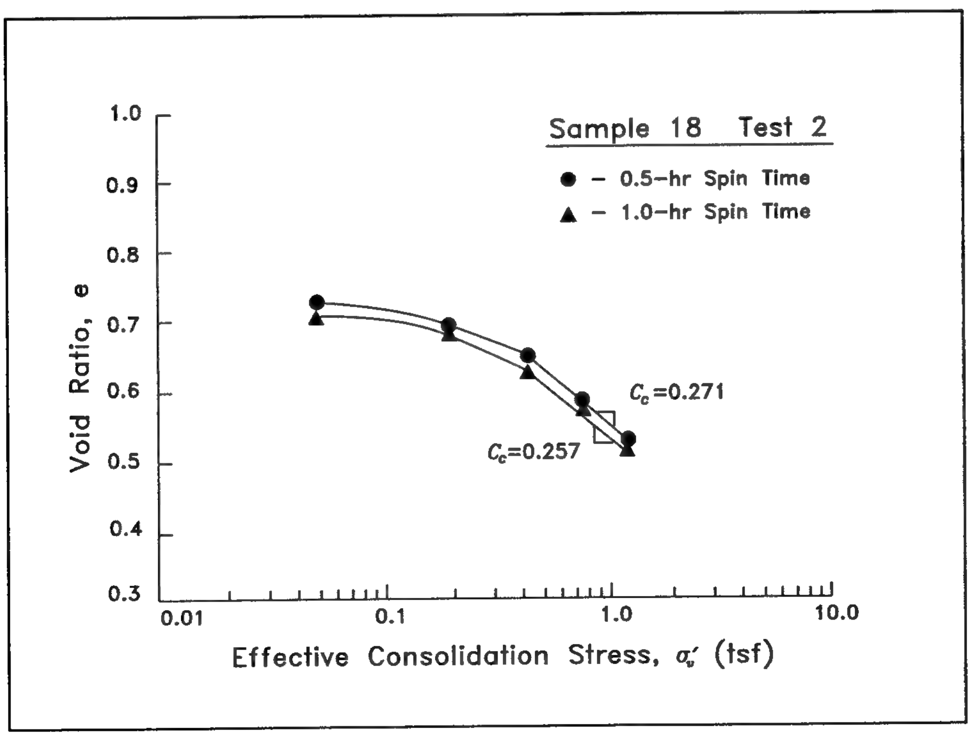

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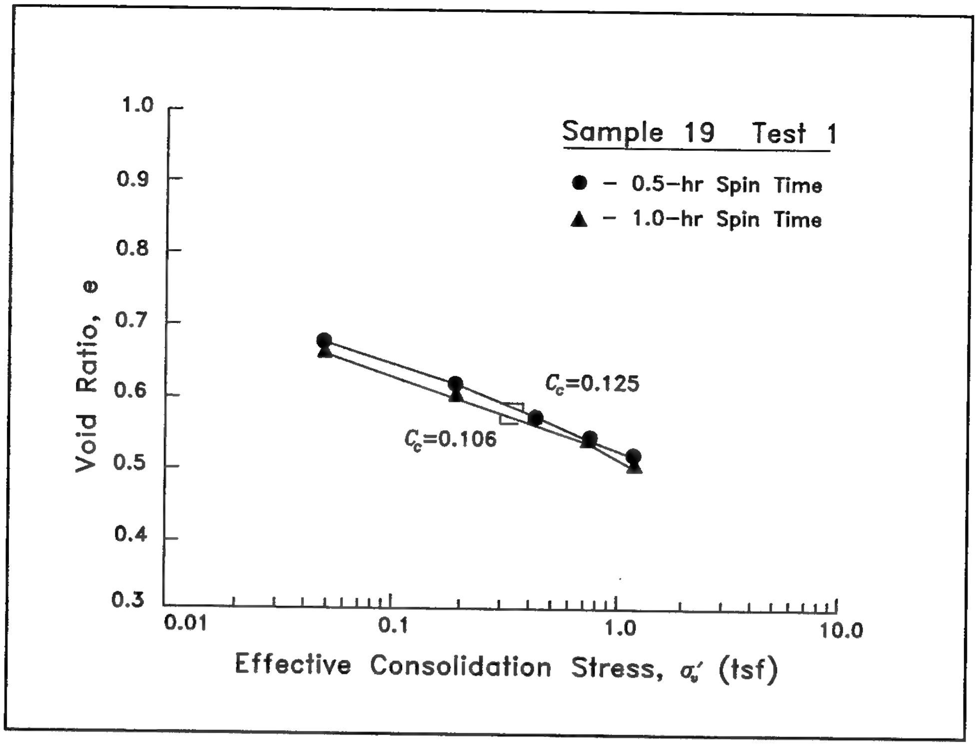

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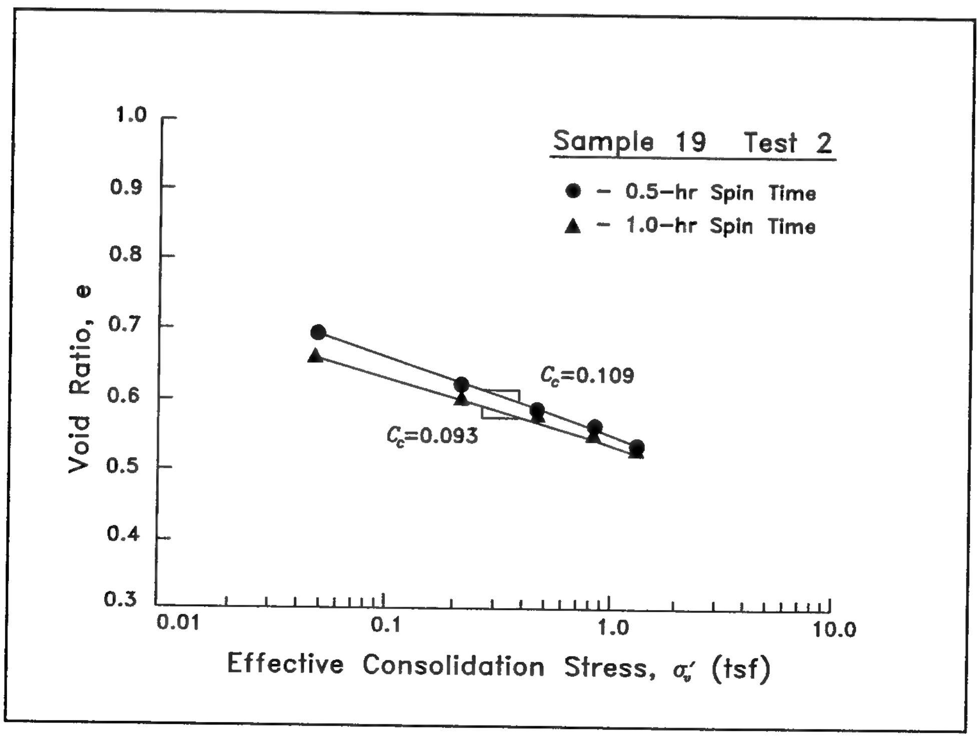

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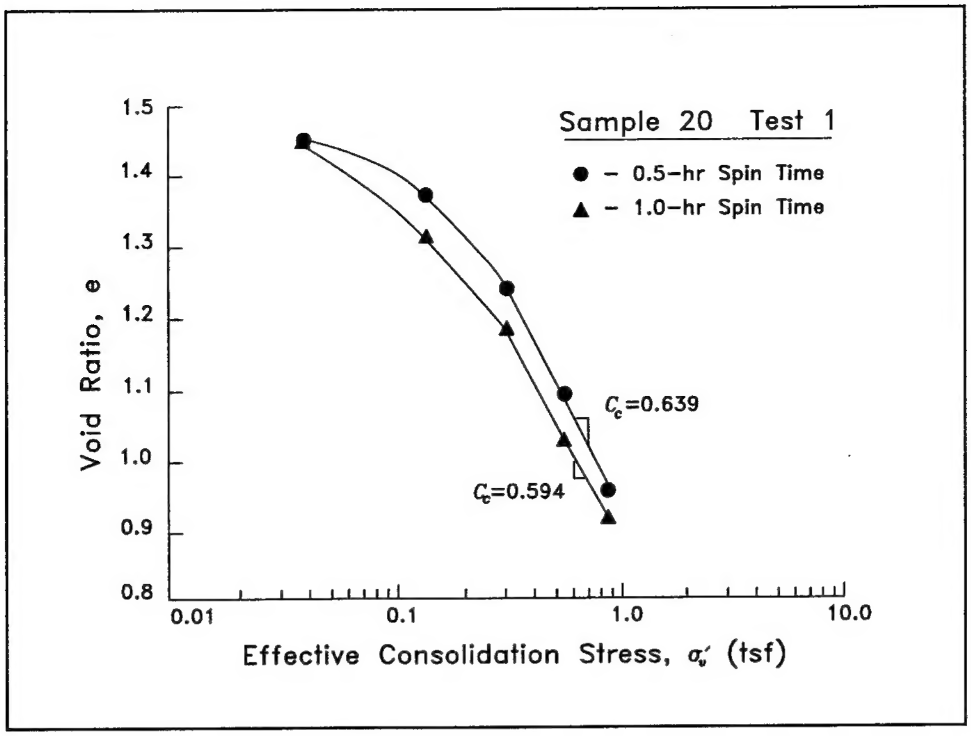

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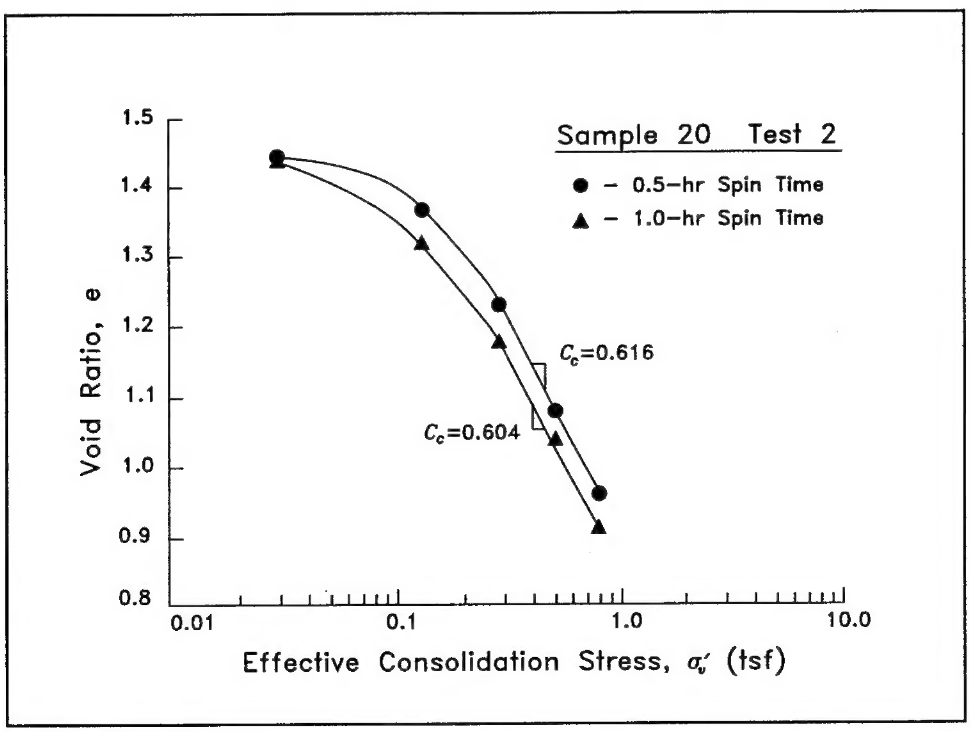

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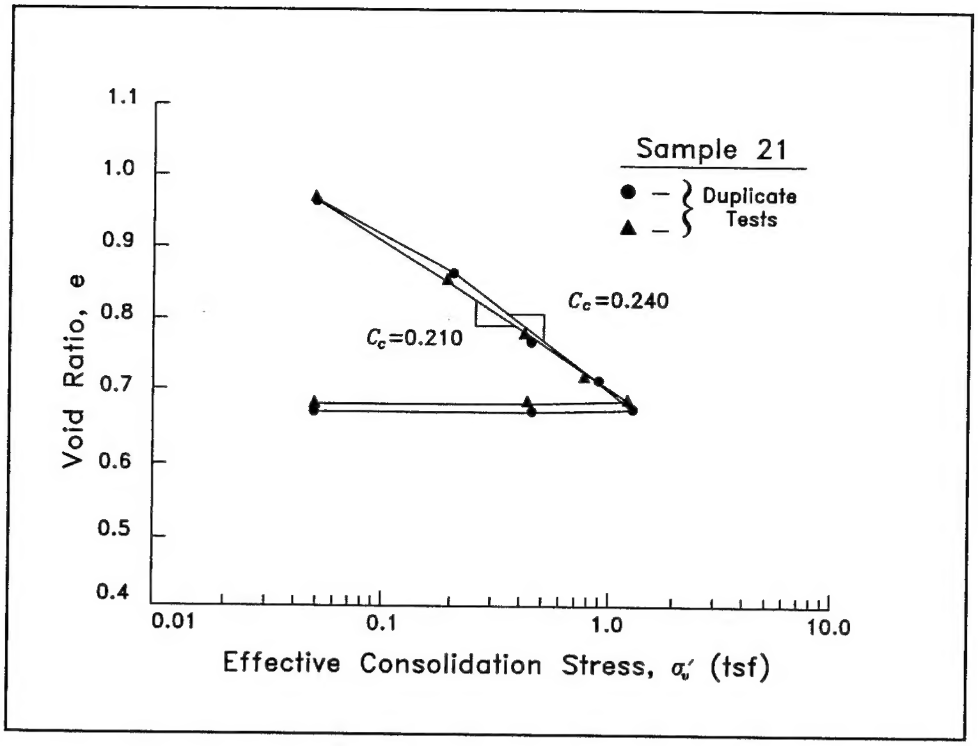

Figure A41. Soil sample 21, 1-hr spin time

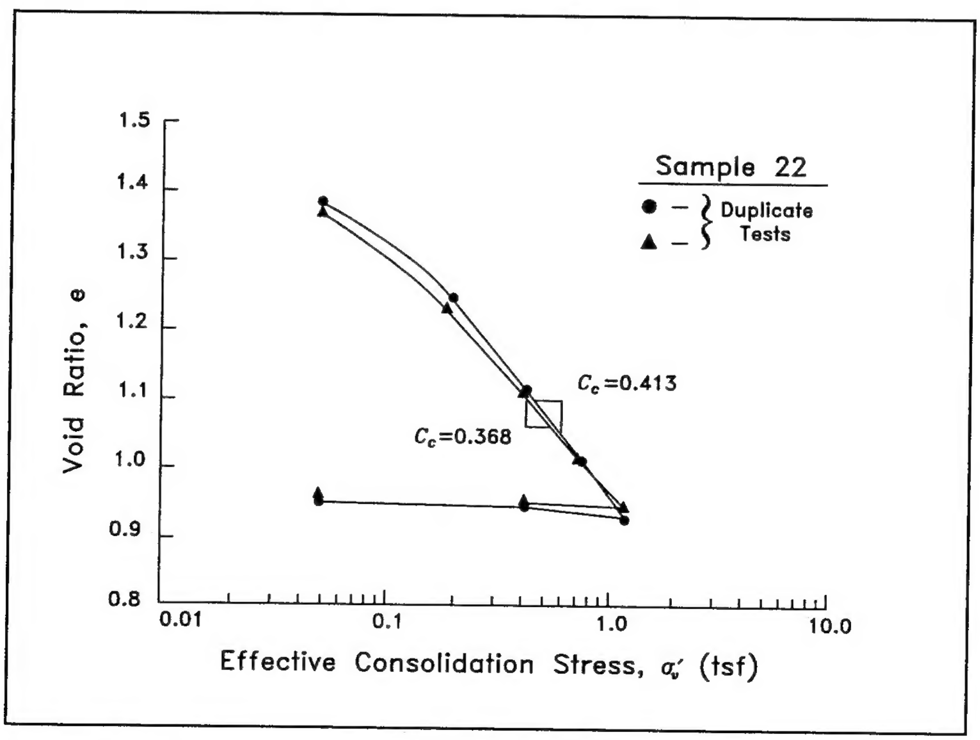

Figure A42. Soil sample 22, 1-hr spin time

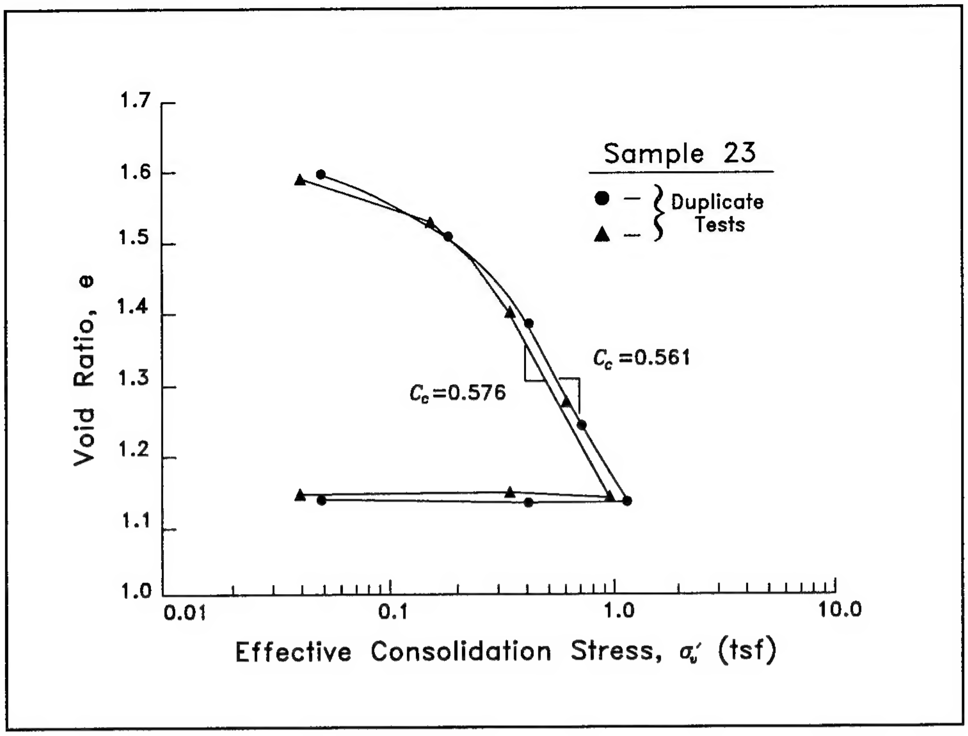

Figure A43. Soil sample 23, 1-hr spin time

*U.S. GOVERNMENT PRINTING OFFICE: 1996-733-144/00032

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

Public reporting burden for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork Reduction Project (0704-0188), Washington, DC 20503.

1. AGENCY USE ONLY *(Leave blank)*

2. REPORT DATE
May 1996

3. REPORT TYPE AND DATES COVERED
Final report

4. TITLE AND SUBTITLE
Centrifugal Consolidation Testing of Soils for Classification Purposes

5. FUNDING NUMBERS

6. AUTHOR(S)
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7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199;
Oklahoma State University, Stillwater, OK 74074

8. PERFORMING ORGANIZATION REPORT NUMBER
Miscellaneous Paper
GL-96-14

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Corps of Engineers
Washington DC 20314-1000

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from the National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

An investigation on the potential for using a laboratory-scale centrifuge for self-weight consolidation of soils is described. Self-weight loading induced by the centrifuge provides a means to establish the soils' virgin compression curves for states ranging from as-sedimented to one ton per square foot effective overburden. While the effects of consolidation time on test results were investigated, time-consolidation properties were not obtained. The principal goal of the investigation was the technique's potential as a classification device based on the well-known correlation between compression properties, undrained shear strength, and Atterberg limits. The technique also shows promise for obtaining compression progerties for dredged materials.

14. SUBJECT TERMS
Centrifuge
Consolidation
Dredged materials
Shear strength

15. NUMBER OF PAGES
63

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
Unclassified	Unclassified		

NSN 7540-01-280-5500

Standard Form 298 (Rev. 2-89)
Prescribed by ANSI Std Z39-18
298-102